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Vice-President, in the Chair.

"The Construction of Deep-Water Quays."

By Alfred Charles Gardner, M. Inst. C.E.

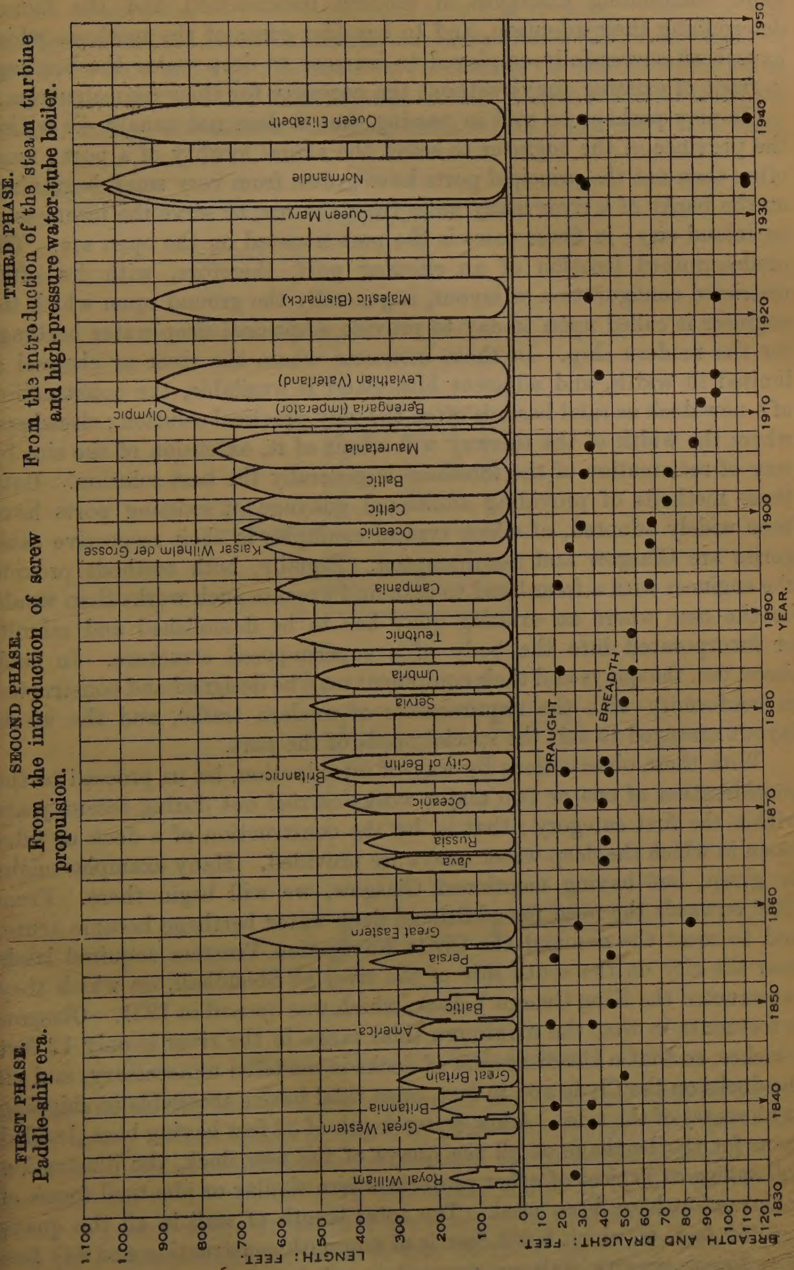
The closing years of the nineteenth century witnessed the opening of a new era in the building of large ocean ships, when dimensions hitherto attained began to be rapidly overtaken, and displacement tonnages greatly exceeded, whilst under the pressure of international rivalry speeds steadily increased in the leading shipping lines of the principal maritime countries of the world. The contest for maritime supremacy at the beginning of the present century was intensified, if not directed, by the revolutionary changes which were then being made in the design of the propelling machinery of many of the newer vessels. The high-pressure water-tube boiler had been developed, and the steam turbine had already entered the field. Added to these outstanding advances in marine-engineering progress, improvements of every kind in the auxiliary services on board ship, and expecially the development of refrigerating machinery, hastened forward the day when the markets of the world began to be served by mercantile vessels of the trading class of dimensions and carrying capacities previously unknown. As an indication of the rapid advances made in the size of ships since that time, it is perhaps sufficient to say that, whereas in 1900 there were no vessels with a gross tonnage of 20,000, and comparatively few between 10,000 and 20,000 tons, by the close of the first decade of the century there were eleven vessels over 20,000 tons and one hundred

* This Lecture was repeated at Meetings of Local Associations at Belfast, Bristol, and Glasgow.

and twenty-five others on the high seas within the lower tonnage range. Many more were either projected or under construction. The latest figures available from Lloyd's Register show that to-day there are eighty-two vessels exceeding 20,000 tons, and four hundred and ninety-six vessels within the range of 10,000 to 20,000 tons, in regular trading service. The continued growth of world tonnage in the matter of large ships is well exemplified by the fact that at the end of June 1939 there were actually under construction no less than fifty-six vessels of from 10,000 to 20,000 tons, four vessels of between 20,000 and 30,000 tons, and three others exceeding 30,000 tons, in the mercantile registers alone. The accompanying diagram, *Fig. 1*, illustrating the principal phases which have marked the evolution and development of the modern steamship, shows at a glance the great progress made in the overall dimensions of the world's leading liners, particularly during the last 40 years. The lower portion of the diagram shows the corresponding advances made during the same period in the draught of such vessels, from which it will be noted that a draught of 30 feet was not attained until the beginning of the century. Since that time it has been reached and exceeded by many vessels other than passenger liners. The increase in warship construction has been scarcely less pronounced, both in tonnage and dimensions.

All these momentous developments in the number and size of ships have conspired to bring about changes no less revolutionary in the political economy of the principal port authorities throughout the world. There was a time, long since past, when ships were built to suit the accommodation of the ports and harbours which they served. That condition of affairs has long been superseded by the building or reconstruction of ports to suit the requirements of ships, and the great advances which have been made in the dimensions and draught of ships have involved in turn changes of an equally revolutionary character in the design of docks and harbours required to accommodate them. It has meant the wholesale reconstruction of many of the older docks, and the deepening and widening of the approach channels to many of the older harbours, and has created problems of considerable magnitude in the way of docking and berthing facilities at all the leading ports. The factor of depth or loaded draught is the one which has given rise to most of the problems confronting the dock engineer to-day, and this factor has reference not only to the passenger liner, but to an ever-growing number of merchant vessels of the trading class.i n which draughts of from 25 to 30 feet are now quite common. The provision of ample berthage accommodation at deep-water quays has, in consequence, become an urgent necessity at every modern port. For the purpose of this Lecture, I shall define a deep-water quay as one in which there is available at the lowest state of the tide a minimum free-water flotation of not less than from 25 to 30 feet. Many such quays have been built in recent years, and various designs and types of construction have been adopted. It is my purpose to describe briefly some of the

Fig. 1.



GROWTH OF OVERALL DIMENSIONS OF OCEAN STEAMSHIPS.

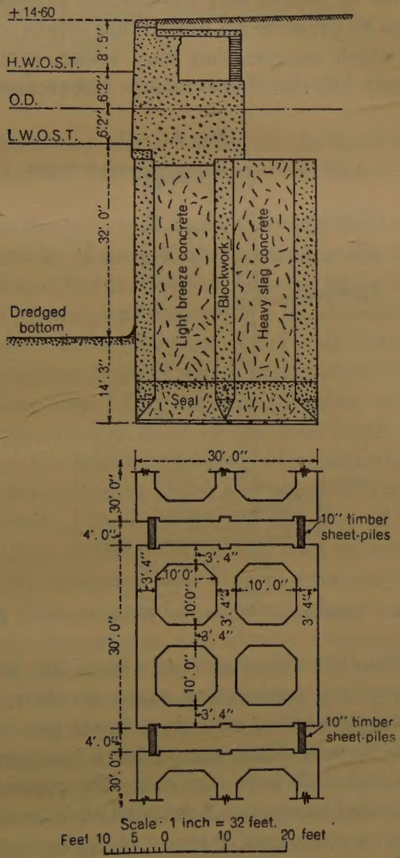
more outstanding examples of modern construction and the factors determining their adoption, and to consider some of the methods which have been successfully employed to increase to deep water the berthage in front of existing quays without the necessity for their reconstruction.

It may perhaps be said in passing that it does not usually lie within the province of the engineer to select the actual locality of a port; more often than not the principal ports have grown from very small beginnings on the banks of a river many miles from the sea, to serve the interests of an inland city; in fewer cases is the port situated on the open sea. The predetermined location of an existing port, therefore, with its often restricted configuration or layout, is generally the ground upon which the engineer is called upon to-day to provide additional deep-water berthage for the modern ship. In those instances where the river or channel is limited in width, and adjacent hinterland is available, the construction of a new dock or tidal basin is often the only practical solution; elsewhere, where the width of the tideway will admit of it, extension of the site by way of reclamation of the foreshore is generally the best solution. Both these methods of providing additional quayage at existing ports have been widely adopted, of which typical examples in their respective categories are Glasgow and Southampton. Equally, both methods provide the engineer with a freer hand to plan and execute such works than would be the case where an existing quay has to be demolished and rebuilt, or incorporated into the body of a superimposed structure. In such cases, too, the quays and their equipment may be designed and constructed in conformity with the requirements of modern traffic, and the whole layout arranged to suit the special needs of the port.

With these observations by way of introduction, let us proceed to the consideration of one or two major works carried out during recent years within the first category; that is, by the construction of a dock or tidal basin in which the deep-water quays are provided. Many examples might be given, but having mentioned Glasgow, we will begin there. From 1920 onwards the need for additional deep-water berthage became acute, and to meet the demand the Clyde Navigation Trustees acquired lands near the city on the south side of the river at Shieldhall, on which they constructed the King George V dock, which was opened in 1931. Glasgow is fortunate in that the maximum tidal range in the river is only 12 feet 6 inches, so that a closed system of docks is rendered unnecessary, and all the docks on the Clyde are, in fact, open tidal basins accessible to shipping at all states of the tide. A low-water depth of 32 feet having been decided upon for the berthage at all the quays of the new dock, the illustration will suffice as typical of other projects where similar or identical forms of construction have been used. The total length of wall in all the quays is 6,450 feet, of which 5,601 feet is of monolith construction and 849 feet of ordinary gravity wall constructed in piled trenches. The monoliths at Shieldhall are 30 feet square, each containing four excavation wells,

and are constructed on steel shoes, each weighing 15 tons. The form and arrangements of the monoliths, as also a cross-section of the gravity wall, are shown in *Figs. 2* and *Fig. 3* (p. 134). In considering the reasons which led to the adoption of these two distinct forms of wall, it is perhaps well to say for the information of Students, to whom this Lecture is primarily addressed,

Figs. 2.

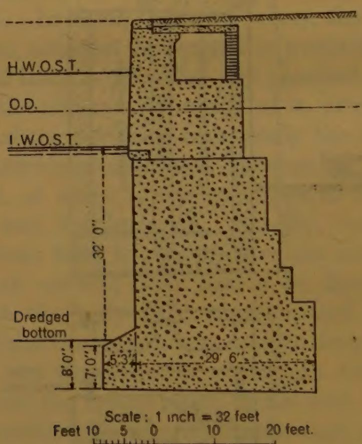


GLASGOW: MONOLITH WALL OF KING GEORGE V DOCK.

that no engineering works of any magnitude should at any time be undertaken, or any detail designs prepared or more than approximate estimates given, until the most exhaustive information has first been obtained as to the character of the ground upon which they are to be built. Especially is this necessary where dock works are concerned. Information, however seemingly unimportant, gathered it may be from apparently uninformed sources, may often prove of value; added to which a little literary research into the ancient history of the district may not be without reward. A

knowledge of the geology of the region, augmented by the more precise information obtained from a system of deep borings and trial pits, not only on, but adjoining, the area of the site, should then give the engineer a fairly reliable idea of what he ought to know. In recent years much closer attention than formerly has been given to the subject of soil-mechanics; that is, the study of the mechanical properties of soils in their relation to questions of lateral and foundation pressures, and the effect of consolidation due to pressure on underlying strata of low permeability, such as clays, in which compression may occur, giving rise to gradual settlement. A study of the more recent researches into this subject

Fig. 3.



GLASGOW: ORDINARY WALL OF KING GEORGE V DOCK.

will well repay the Student whose activities lie in the direction of designing or carrying out heavy civil engineering works involving deep foundations. In the case under consideration, numerous bores put down on the line of the projected quays showed that, except for a comparatively small area, where boulder clay and rock were met with, the greater part of the walls would have to be founded on sand. A detailed examination of the materials from these bores, and a comparative study of the boring records, indicated that monolith construction would be the most satisfactory method, except in those areas where the rock and boulder clay were encountered, in which areas the solid gravity wall was constructed.

The general conditions governing the design of gravity walls have been very fully dealt with in numerous works on the subject. The subject is rather a complex one, and involves a number of considerations not always capable of exact determination; as affecting the stability of a wall, however, the principal forces acting upon it are:—

(1) The horizontal pressure on the back of the wall, due to the

material forming the filling, and any water which may be contained in the ground.

(2) The horizontal pressure due to surcharge or superimposed loading on the ground behind the wall.

(3) The pull of the ship's mooring on the quay-front at its highest point.

These constitute the principal overturning forces.

Resisting these forces are :—

(1) The vertical weight of the wall itself, and any direct loads upon it.

(2) The weight of any filling resting on the back steps of the wall.

(3) The unbalanced water-pressure on the face of the wall.

These constitute the principal forces resisting overturning.

In addition, there is, of course, the resistance due to friction between the base of the wall and the underlying material to be taken into account, which is the factor of principal importance in preventing sliding, as well as the upward pressure under the base of the wall due to buoyancy where it rests on a porous strata, both of which are fundamental considerations.

Other calculations depending on design arise when, for example, there is a toe or projection in front of the wall itself, but in the general case the stability of the wall is attained when the algebraic sum of the moments of all the forces acting about any point in it is zero.

In practice, however, the location of the line of resultant pressure within the middle third of the width of the wall at dredged level or dock bottom will generally ensure a safe margin of stability, and by general assent this has long been accepted as a safe working rule.

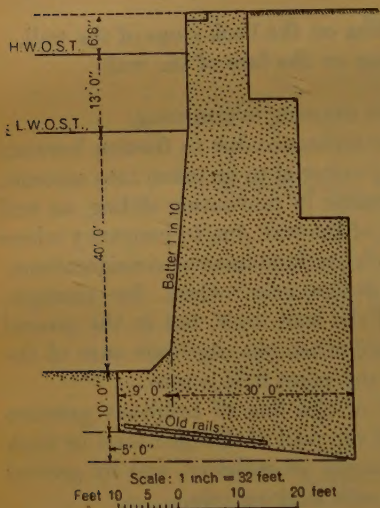
The indeterminate factors are, and always must be, the nature of the ground upon which the wall rests, and the behaviour, characteristics, and water-content of the material constituting the filling behind the wall. For this reason, the effective drainage of the back of the wall is a matter of considerable importance in relieving the back pressure by effecting a balance between it and the varying tidal pressure on the front of the wall.

Provision against the possibility of sliding (where there is any reasonable likelihood of this occurring), which has been the most fruitful source of failure in walls, can best be secured by sloping or stepping the base of the wall downwards from front to back, or by driving sheet-piling in front of it. Every case, however, must be determined on its merits, and in the light of all available information, not only concerning the character of the ground upon which the wall rests, but also with regard to the underlying strata for some considerable distance below it. Information on this point is of equal importance with that of the actual bearing strata, for the reasons previously referred to. The accompanying diagram (Figs. 4, Plate 1) illustrates the conditions obtaining in the case of the monolith

wall at Shieldhall dock, in which the principal factors referred to have been taken into account.

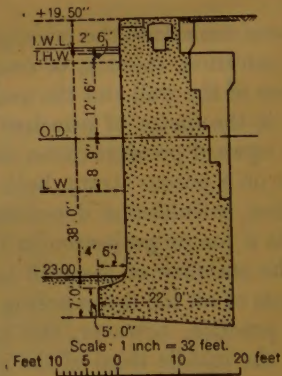
It was perhaps only to be expected that a period which witnessed such a remarkable increase in the size and tonnage of ships as the last 40 years, should have been marked by the adoption, wherever practicable, of the more solid forms of construction associated with the gravity wall or its later development, the monolith, in preference to lighter forms of construction such as the piled or open-stage quay. There was much to be said in its favour, especially in view of the concurrent advances in the

Fig. 5.



SOUTHAMPTON: WALL OF OCEAN DOCK.

Fig. 6.



LONDON: NORTH QUAY WALL OF KING GEORGE V DOCK.

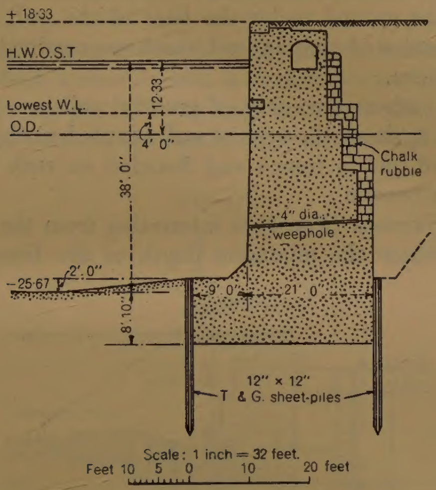
dimensions and weight of quayside cranes and dock equipment generally, both in respect of its robustness and low maintenance costs, with the result that many excellent examples of this type of wall are to be found in the deep-water quays built during the last 25 or 30 years in all parts of the world.

The wall of the Ocean dock at Southampton, the walls of the King George V docks at London and Hull, and those at Liverpool, Swansea, and Immingham, to mention only a few instances in Great Britain, are all typical illustrations of this form of construction, exhibiting in their several contours variations and divergencies imposed by the special conditions of each case.

It will be noted, in the case of the Southampton wall (*Fig. 5*)—the first of the series here referred to—that the width of the wall at its base is more than half its height, whilst its depth below the bottom of the dock presents

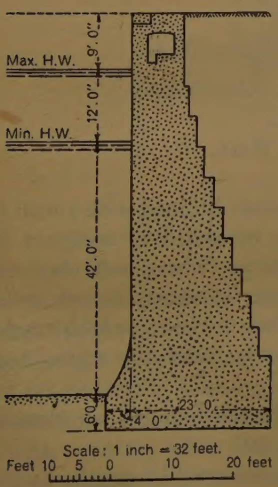
a continuous vertical pressure-face 10 feet in height opposed to the resistance of the earth. The character of the ground upon which this wall

Fig. 7.



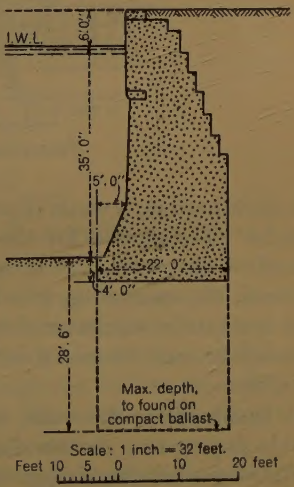
HULL: WALL (ON RUNNING SAND) OF KING GEORGE V DOCK.

Fig. 8.



LIVERPOOL: WALL OF BRANCH DOCK, GLADSTONE DOCK.

Fig. 9.



SWANSEA: WALL OF KING'S DOCK.

is built consisted of a particularly plastic clay, and the history of the failures due to sliding which followed the construction of the walls of the

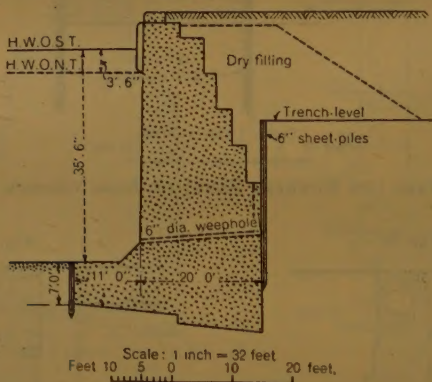
earlier Empress dock in that vicinity suggested the dimensions of width and foundation depth illustrated in the cross-section.

The walls both at London and Hull (*Figs. 6* and *7*) follow more normal proportions. In the former case they were founded on ballast and chalk, whilst at Hull the ground under the base of the wall in the particular section given consisted of running sand which was confined between sheeting in the manner shown.

At Liverpool, where a variety of gravity walls are to be found, the illustration (*Fig. 8*) shows the type of wall adopted at the Gladstone dock, which was carried out in trench and founded on rock and gravel some 6 feet below dock bottom.

The wall at Swansea (*Fig. 9*) is interesting from the fact that in the section shown, where the minimum depth of the foundation is 4 feet

Fig. 10.



IMMINGHAM: DOCK WALL.

below dock bottom, a water depth of 35 feet is obtained with a wall 45 feet in height. The ground for the most part consisted of compact ballast of excellent character, but in some few places, where soft clay was encountered, the excavation trenches were carried down deeper, attaining in one instance a maximum depth of 28 feet 6 inches, which made the height of the wall 69 feet 6 inches, or nearly twice the water depth in front of it.

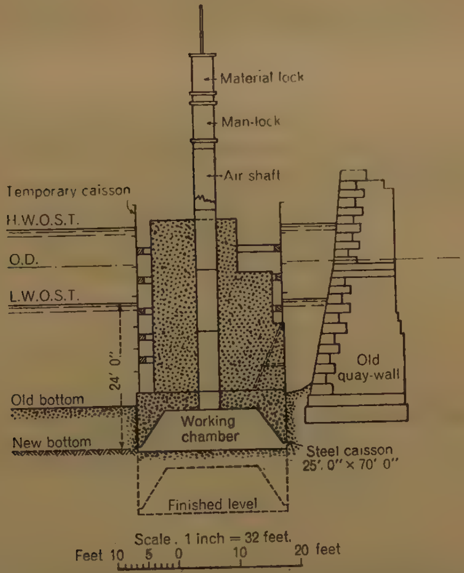
At Immingham, the walls, where they were founded on clay, are of the wide sloping-base section shown in *Fig. 10*.

Where a solid foundation exists at a reasonable depth, and open-trench excavation can be adopted, the gravity type of wall is generally the most economical form of construction, especially where the loads on the quays are considerable, or the wall is required to accommodate heavy moving structures such as travelling elevators or coaling cranes. There is a point, however, where this type of construction ceases to be economical,

or even practicable, and recourse must be had to other methods; that is, either to a wall founded on caissons, or to one composed entirely of monoliths carried down to strata of sufficient strength and thickness to carry the weight, or else to some form of braced structure involving the use of piles.

The caisson wall, using the term in its strictest sense, is that form of construction in which a steel caisson is sunk under compressed air, and a gravity wall built up on it as sinking proceeds. The caisson itself is usually of the same width as the base of the wall, although it may be wider;

Fig. 11.



GLASGOW: SINKING CAISSON WALL OF PLANTATION QUAY.

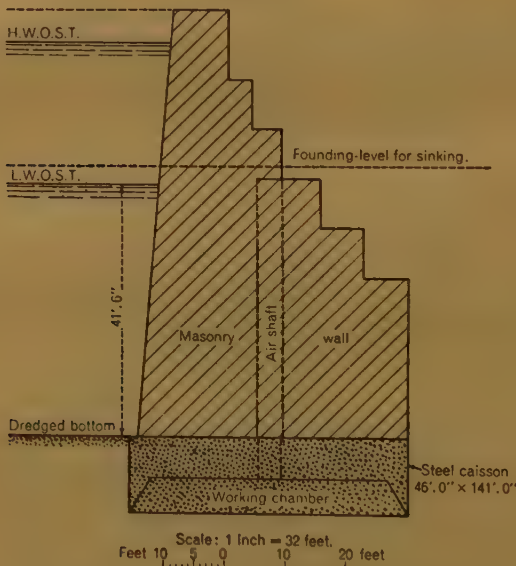
and is usually rectangular in form, and of a length adapted to the conditions and requirements of the site. The working chamber must be of sufficient height to permit the excavated material to be removed through vertical access-shafts carried upwards through the wall to air-locks above high-water level, the working chamber and the access-shafts being finally sealed with concrete when founding-level is reached.

Several such walls have been built in Glasgow harbour in recent years, of which a typical section is given in *Fig. 11.* Here it will be noted that the caisson is somewhat wider than the wall itself, and was so designed to permit the pressure on the foundation to be kept within reasonable limits as well as to equalize the loading during sinking. These caissons were built up on a piled staging and lowered into position on a previously-

dredged bottom. Perhaps the largest caissons used in this way are to be found in the north wall of the new basin at Havre, where rectangular steel caissons 141 feet long and 46 feet wide, built ashore and floated into position, were sunk to a depth of about 12 feet below dredged level, on which a stone-faced mass-concrete wall was constructed, as shown in *Fig. 12,* thus providing a deep-water berth of about 42 feet at low water of ordinary spring tides.

The caisson method, involving as it does excavation by hand, or, at the best, by means of mechanical spades, requiring the material so

Fig. 12.



LE HAVRE: CAISSON WALL OF NORTH QUAY.

removed to be lifted through access-shafts and discharged through air-locks, is neither an expeditious nor an economical one, but it is sometimes the only method that can safely be adopted to reach a foundation through any considerable depth of soft mud or running sand, or for the construction of an isolated quay in open water where there is any considerable rise of tide. One of the chief advantages of the caisson is the comparative ease with which it can be controlled during sinking and ultimate founding.

The more general method adopted to-day for the construction of deep-water quays is the monolith system. The term "monolith" is more strictly applicable to that type of wall in which separate and independent structures of brick or concrete are built up in situ on specially-designed bases, enabling them to be sunk by their own and superimposed weight by means of grabs operating through one or more interior wells. A line of such structures,

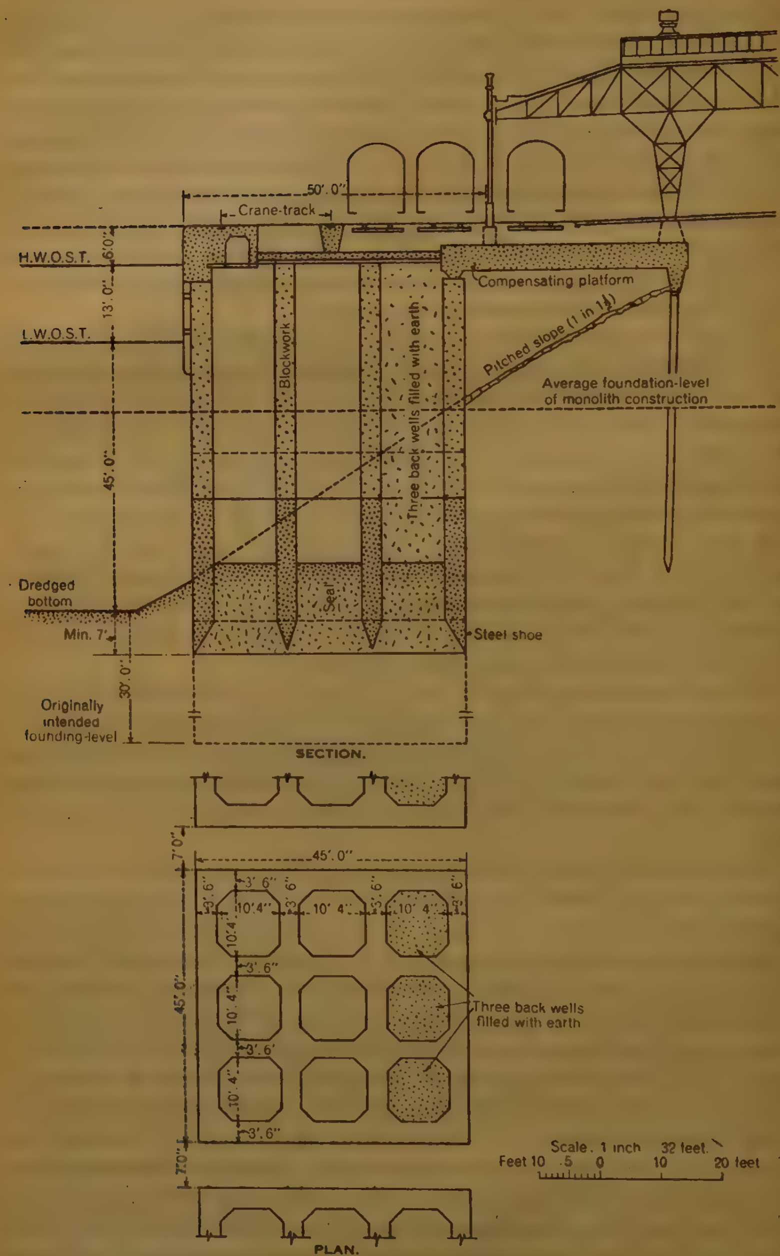
which are virtually independent towers in close proximity to one another, thus constitutes the wall. In principle, the monolith is a development of the cast-iron cylinder commonly used in bridge and pier foundations, and may just as consistently be sunk by excavating under compressed air as by open grabbing. The essential feature is that it is a single self-contained and self-supporting unit in the structure of the wall. In its simplest form the monolith in plan is a hollow square or rectangle with a single interior well, but more usually, in the massive construction required for a deep-water quay, it is divided into four or more wells strengthened by internal walls of considerable thickness. The shoe or base upon which it is built up may be of timber, steel, or ferro-concrete, wedge-shaped in section to provide the requisite cutting edges.

If they are constructed of concrete, the monolith walls may be built up as sinking proceeds, by pouring the concrete in lifts or stages between steel or timber forms, the shuttering rising as the sinking advances; alternatively, they may be built up in courses of pre-cast concrete blocks. The latter method is the one generally employed today, for the reason that it lends itself more readily to the placing and disposition of the weight at points where assistance in sinking is required. To ensure rigidity, the lower part of the monolith for some height above the base is usually carried out in lightly-reinforced mass concrete by the former method, the block courses continuing thereafter, either keyed or bonded together horizontally, and held in position vertically by steel rods carried upwards at intervals through the courses from the base.

One advantage attending the use of monolith construction in the case of a deep-water quay is the facility which it affords for relieving tension at the back of the wall by omitting altogether the filling above the bottom seal in the front wells. This is a point of considerable importance where the wall is of exceptional height, and has been adopted in a number of cases.

The monolith form of construction was employed throughout the building of the new deep-water quay at Southampton, which is perhaps the finest example of its kind yet completed. The work at Southampton is also an instance of additional deep-water berthage being provided at an existing port by the reclamation of the adjoining foreshore, in this case to the extent of some 400 acres over a frontage of approximately 2 miles. The quay-wall itself has a length of over 7,500 feet, and is designed to provide a depth of 45 feet of water at low water of ordinary spring tides. It consists of one hundred and forty-six monoliths, each 45 feet square in plan, and containing nine interior wells, as shown in *Figs. 13* (p. 142). It is thus one of the largest works carried out on this principle, designed to accommodate some of the largest passenger liners in the world. An interesting feature of the design applying throughout the greater part of the length of the wall is illustrated on the cross-section, in which a massive reinforced-concrete table or compensating platform is introduced, the front edge of which

Figs. 13.



SOUTHAMPTON; MONOLITH WALL OF TEST QUAY.

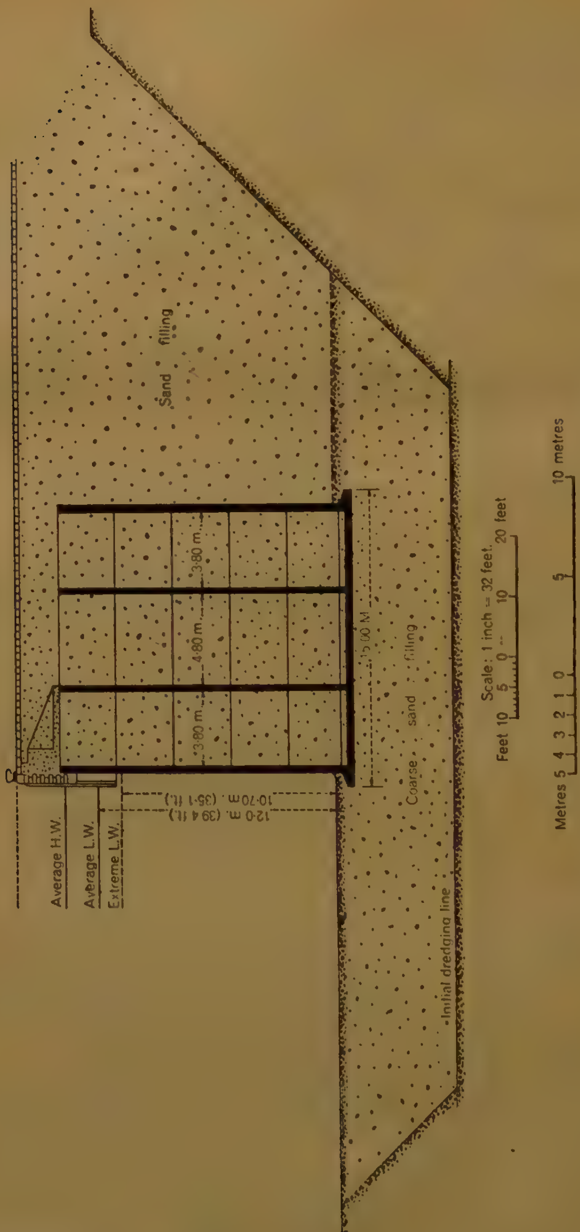
rests on the back of the monolith, its inner edge being supported on reinforced-concrete piles. The reason for its introduction is explained by the fact that the difficulties attending the sinking of the monoliths without damage through the lower strata of greensand were so great that they were founded at some distance above the originally-intended level. The loss of stability resulting from the reduced weight of the monolith was thus compensated in the ingenious manner shown.

Whilst the monolith system of construction has been most generally adopted by British engineers, other forms of construction, some of them quite unique in character, have found favour in Continental practice. Amongst such works rendered necessary for the accommodation of the modern deep-draughted liner, few deserve more prominent mention than the work carried out by French engineers in the port of Havre. Figs. 14, Plate 1, show a method of construction adopted on the north quay of that harbour, where a composite wharf was constructed for the berthing of these vessels, providing a low-water depth of 45 feet. This work is particularly interesting from the fact that its construction virtually consists of three portions, more or less separate and distinct. The actual berthing face consists of a series of square dolphins, spaced 140 feet apart, against which the vessels lie. These are built-up reinforced-concrete boxes, of a total height of 82 feet, and sunk to a depth of 1 metre (3 feet $3\frac{3}{8}$ inches) below the dredged level of the basin. They are formed with nine internal chambers, in each of which a reinforced-concrete pile is driven well into the ground, the compartments thereafter being filled with concrete. At a distance of 16 metres (52 feet 6 inches) from the back of this line of dolphins, an L-shaped retaining wall was built, supported on an independent system of piling of its own, the retaining wall itself being held back by steel tie-rods to anchorages in the rear. Between the dolphins and the retaining wall, in the 16-metre space mentioned, the pile-supported wharf or decking is constructed, independently of both the dolphins and the retaining wall. A radial system of ties connects the dolphins with the retaining wall below deck-level. Figs. 14, Plate 1, show the disposition of the tie-rods and piling in the wharf, and the system of strut-bracing between the dolphins and the retaining wall. This work, quite unique in character, has the merit of possessing a certain degree of flexibility, combined with lightness of construction, a quality not without value where the berthing of such great ships as the *Normandie* and her consorts is concerned.

Another form of construction successfully adopted in loose ground is to be found at Rotterdam, Dordrecht, and other Netherland ports. The western area of that country, owing to its peculiar alluvial formation, and to the character of the material constituting the estuarial lands of two great rivers, has presented problems of peculiar interest to the engineers called on to construct harbour works upon them.

Over the whole of this area, and to a considerable depth below cultivated-ground level, the subsoil consists of layers of clay and bog, of very

Fig. 15.



ROTTERDAM: QUAY FOR WATER-DEPTH OF 12 METRES.

limited bearing capacity, but below this, at depths ranging from 30 to 40 feet, is to be found a deep stratum of sand, sometimes mixed with gravel. It is upon this foundation that the deep-water quays of Rotterdam have been built.

The construction of such works in ground of this character has been achieved by first of all dredging away the loose and inferior material overlying the substratum of sand at the place where the walls have been erected, and replacing by depositation in the deep areas so dredged a firm stratum of coarse sand or ballast, on which, after consolidation and levelling, pontoons or closed caissons are founded. The initial step has therefore been the improvement of the ground by artificial means. *Fig. 15* shows a cross-section of the latest form which this method of construction has reached, on which the initial dredging line is indicated, and the filling shown. The pontoons themselves, which consist of reinforced-concrete boxes, are floated into position, and lowered on to a bed levelled off by dredging to a depth of 1 metre below the intended dock bottom. The low-water depth available in front of the quays at Rotterdam is about 40 feet, and the successful completion of these works is deserving of mention as the solution of a problem unique in character, which has been repeated successfully in other parts of the world.

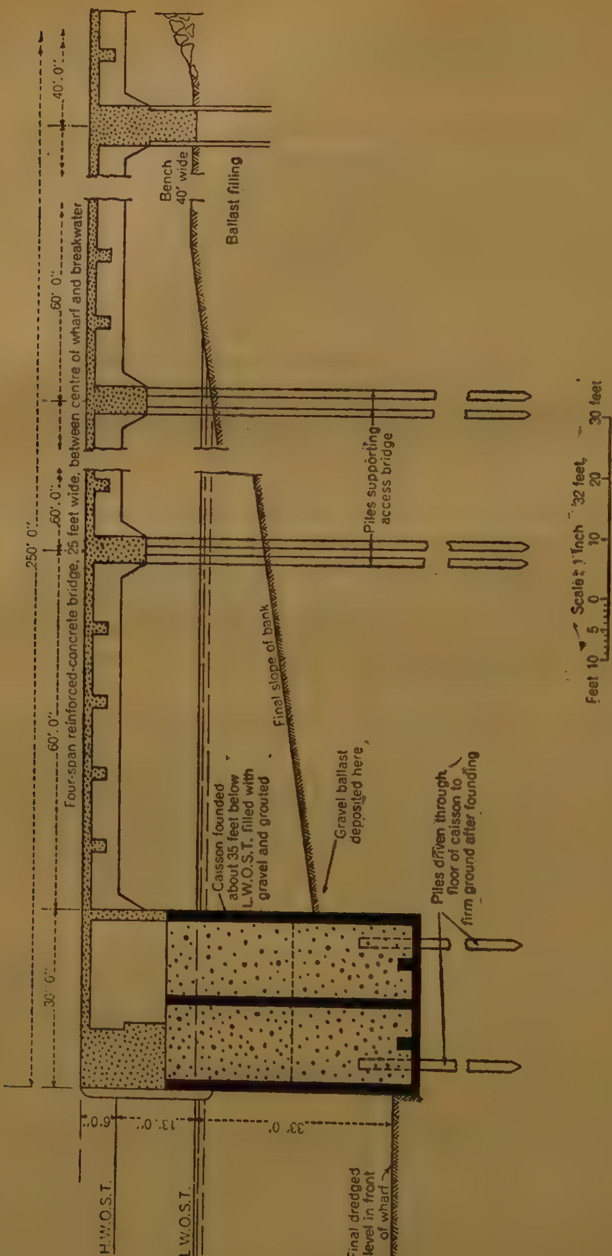
In the earlier works carried out with this type of wall, mass concrete was used as filling in the compartments of which it was composed, but this has since given place to sand filling only, with resulting substantial economy.

The question of stability in these walls is a matter requiring very careful investigation, introducing, as it does, features not met with in the ordinary gravity wall. The engineers responsible for the design at Rotterdam state that the stability is considered satisfactory when the resultant of all the forces acting on the wall cuts the ground-area of the pontoon within the centre compartment, provided that it forms no greater angle than 30 degrees with the vertical and that a foundation-pressure of 40 tons per square metre is not exceeded.

Examples of the same or similar forms of construction are to be found at Marseilles, Naples, and other European ports, and as far afield as Japan, whilst in the British Isles extensive use has been made of this system in the building of some of the modern quays in the port of Dublin. A section of a deep-water quay now under construction in that port is shown in *Fig. 16* (p. 146).

The port of Copenhagen furnishes somewhat unusual forms of construction in the quay-walls which have been built there for the accommodation of modern liner traffic. The new basin recently constructed to the north of the Free Port provides berthage for such vessels to a low-water depth of 10 metres (32 feet 10 inches). For part of their length these quays are constructed of reinforced concrete, the face of the quay being formed by a continuous line of interlocked sheet-piling surmounted by a slab

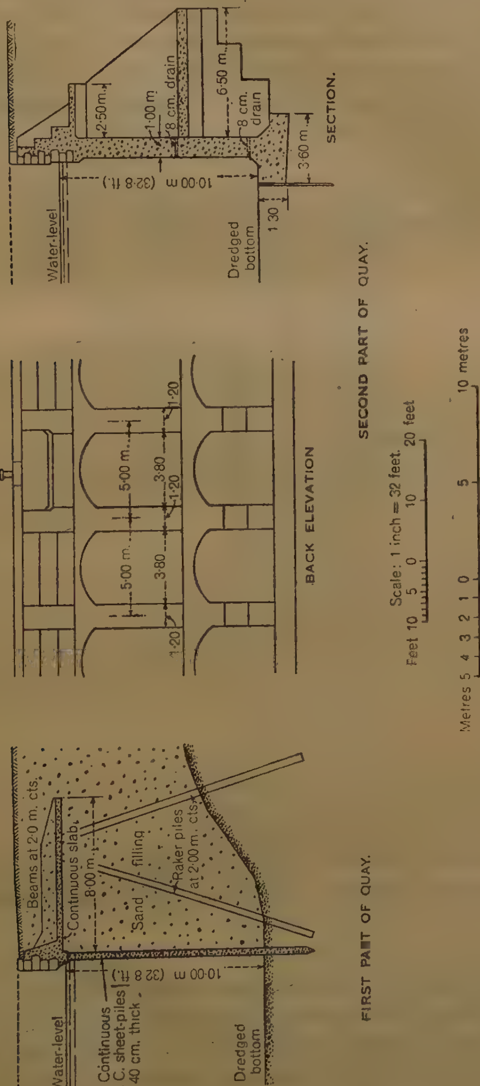
Fig. 16.



DUBLIN : WHARF AT EASTERN BREAKWATER.

supported at the back on raker piles, as shown in *Figs. 17.* The remainder of the length of these quays consists of buttressed concrete walling of the

Figs. 17.



COPENHAGEN: QUAY FOR WATER-DEPTH OF 10 METRES.

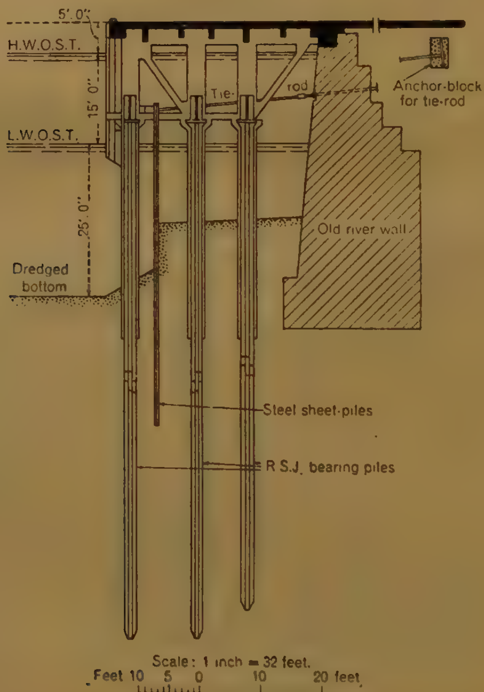
form shown in the second part of the diagram, where conditions necessitated the construction being carried out behind the protection of temporary cofferdams.

Other methods of obtaining additional deep-water berthage in existing

ports have been successfully employed, by constructing new or false quays in front of the older works without the necessity for their demolition.

An interesting work falling within this category has been carried out within recent years by the Tyne Commissioners, who have constructed a deep-water quay in front of an existing river-wall. The new quay, which is 1,100 feet in length, encroaches into the river an average distance of about 37 feet over the greater part of its length. A typical cross-section

Fig. 18.

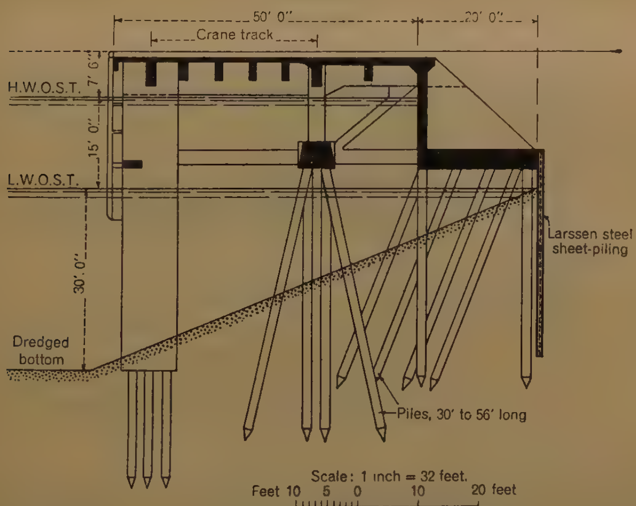


North Shields: North End of Tyne Commission Quay.

of the work is shown in *Fig. 18.* Here it will be noted that interlocking steel sheet-piling has been driven some distance in front of the old wall to support the ground in front of it, the top of the sheeting being held in position by tie-rods through the old wall to a line of concrete anchor-blocks well to the rear. The necessity for securing the old wall in this way by means of sheeting in front and tie-rods assisting in its support was a wise and necessary precaution, which has been adopted in several such cases where doubts existed regarding the stability of the wall. In one section of this work, where it was found possible to secure a foundation on the sandstone or the underlying shale bed, the quay was founded on

cylinders. These cylinders were of 13 feet and 6 feet diameter respectively, sunk under compressed air and concrete-filled. At other parts of the work great difficulty was experienced in finding a suitable founding-depth, and the tests carried out indicated that piles would need to be driven to at least 75–80 feet below low-water level in order to find a bottom. In the end, it was decided that steel piles of cruciform section would alone meet the requirements, and the piling used in the cross-section shown (*Fig. 18*) consisted of rolled steel joists, the main joist being 20 inches by 7½ inches, with smaller joists of 12-inch by 6-inch section riveted at right angles to it on each side of the web. Above the bed of the river these piles were encased in a 3-foot diameter steel tube filled with concrete.

Fig. 19.

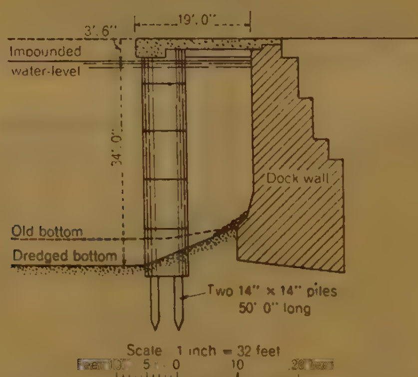


NEWCASTLE: QUAY EXTENSION.

This example is given as showing a very ingenious method of meeting what were undoubtedly great difficulties, which by this means were successfully overcome. Another example on the Tyne of a deep-water quay is that shown in *Fig. 19*. Here it will be noted that the quay, which is also on the river-front, is an independent ferro-concrete structure in which the piling under the L-shaped retaining wall and on the centre-line of the quay is sharply raked towards the river—a very necessary precaution, having regard to the character of the ground, and to the fact that local conditions did not permit a system of tie-rods and anchor-blocks to be provided to the rear of it. The front group of piles, six in number, are encased in reinforced-concrete cylinders having a diameter of 9 feet, whilst the ground at the back of the structure is confined behind a continuous line of sheet-piling bearing against the structure itself.

Few port authorities have felt the necessity for increasing the deep-water accommodation in their docks to a greater degree than the Port of London Authority, who have recently carried out a very interesting piece of work on the north quay of the Royal Albert dock. This quay was originally designed for a depth of 27 feet below the impounded level of Trinity high water, but it became necessary in 1911, in consequence of the increasing draught of ships, to increase the available depth there. This was done in the first instance by raising the impounded-water level in the dock 2 feet 6 inches by the installation of costly pumping machinery, but owing to the continued increase in the size and draught of vessels, and to the number of ships which had to lighten before proceeding to their berths, the engineers at length decided to deepen the berths by dredging

Fig. 20.



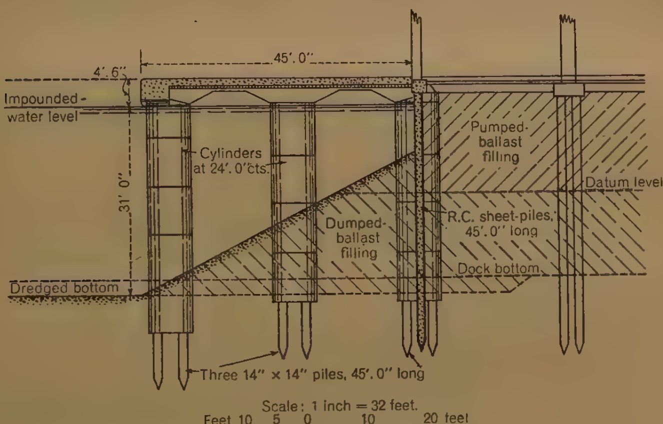
LONDON: NORTH QUAY OF ALBERT DOCK.

out the dock to give a depth of 34 feet. Because of the limited width of the dock, and to avoid undermining the foundation of the existing walls, the construction of a false quay was decided upon, which was carried out in 1935–36 in the manner shown in *Fig. 20.* It will be noted that the false quay is only 19 feet wide, and consists of a reinforced-concrete platform carried on a deep cope-beam supported on cylinders spaced 24 feet apart and tied into the existing quay-wall by transverse beams. The cylinders themselves are pre-cast concrete tubes, within each of which two 50-foot piles were driven down into the dock bottom, the cylinders afterwards being filled with concrete deposited through the water. This work illustrates a very simple and effective way of obtaining the requisite deep-water berthage with a minimum of encroachment on an existing water-area, whilst at the same time securing the safety of an existing wall.

Another form of construction successfully employed in deep-water quay design is that recently used in the reconstruction of the north quay of the Victoria dock, London, a cross-section of which is illustrated in

Fig. 21. The new quay, which is 3,250 feet in length, occupies the site of the once familiar jetties which were for so long the characteristic feature of this dock. It is therefore not perhaps so much a reconstruction as an entirely new work, rendered all the more interesting on account of the demolition work it entailed, and the preparation of the site. It will be seen that the quay itself consists of three rows of pre-cast concrete cylinders, tied together above water-level with heavy transverse beams, which, as portal bracings, constitute the only cross-ties in the structure. The quay is therefore a self-supporting structure, with a considerable table-width.

Fig. 21.



LONDON: NORTH QUAY OF VICTORIA DOCK.

The work involved, in the first place, the reclamation or filling of the open-water spaces between those portions of the jetties left standing to the rear of the new wall. This was done by depositing a ballast bank up to or above datum-level, in which bank the construction works were carried out. Afterwards, the ballast filling was pumped in to quay-level, and to retain this material a curtain wall of ferro-concrete sheet-piling was driven between the main cylinders in the back row. All this material was excavated from the deepening of the dock itself.

As a form of construction, it is somewhat unusual in character, and possesses a robustness more nearly approaching that of a gravity wall than could have been obtained by an open braced structure. It has the further merit of reducing the under-water work to a minimum, and, as a departure from the normal type of mass construction, effects a considerable economy in design.

These are but a few of the more modern and outstanding examples of deep-water quay construction; others might be added in which various

forms of steel sheeting have been employed—usually in a secondary capacity—and others in which open ferro-concrete staging has been adopted on lines similar to the forms employed in braced timber structures, but in general it may be said that mass, rather than lightness of construction, continues to be the main criterion of value in all the more important dock-engineering works.

A vote of thanks to the Lecturer was proposed by Mr. Raymond Carpmael and seconded by Mr. D. F. Orchard, and was carried by acclamation.

JOINT INFORMAL MEETING

WITH

THE INSTITUTION OF MECHANICAL ENGINEERS,

THE INSTITUTION OF ELECTRICAL ENGINEERS,

AND

THE INSTITUTE OF WELDING.

11 March, 1940,

at the Institution of Electrical Engineers.

JAMES ROBERT BEARD, M.Sc., Vice-President I.E.E., in the Chair.

"Emergency Repairs to Plant, with Special Reference to Welding."

Introducer: STANLEY FABES DOREY, D.Sc., M. Inst. C.E., M.I. Mech. E., M. Inst. Welding.

The Introducer, after referring to the numerous applications of welding in the construction of mechanical and electrical plant, stressed the necessity for employing a different technique in the repair of such plant, and pointed out that repair work might be divided into two classes, namely, emergency temporary repairs and permanent repairs under emergency conditions. In the case of land plant a portable welding set was necessary, as well as a stock of suitable steel sections. He then referred to some of the types of repair to which welding could be applied, including water-jackets of engine-cylinders, engine-bedplates, and combustion-chambers, leaking seams, and tubes in boilers. He showed a number of lantern-slides illustrating fabricated machine-parts that would formerly have been made of cast iron. Dr. Dorey concluded by enumerating the points that had to be considered to ensure that a welded repair would be satisfactory, including the accessibility of the work, the weldability of the material, the necessity for heat-treatment or pre-heating, and means of testing the finished weld.

Mr. E. S. Needham (on behalf of the Institution of Civil Engineers) confined his remarks to the repairing of contractors' plant, and to the repair and strengthening of bridges and of points and crossings for railways and tramways.

Mr. C. H. Davy (on behalf of the Institution of Mechanical Engineers) referred more particularly to the precautions necessary to ensure that the right method of welding was employed.

Major James Caldwell, J.P., (on behalf of the Institution of Electrical Engineers) described the application of electric welding to various types of repair, and showed lantern-slides illustrating portable welding plant and repairs to marine engines.

Other speakers referred to the early history of electric welding, to the necessity for the welder to have a knowledge of the properties of the material being welded and of the correct technique to be employed, and to the carrying out of particular repairs, including a steam-separator, boiler-drums, and a turbine-casing. The necessity for co-ordinated research was also pointed out.

The speakers in the general discussion were Messrs. Ramsay Moon, G. B. Plows, E. Kilburn Scott, and W. A. Stanier.

ORDINARY MEETING.

19 March, 1940.

SIR CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A., President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Reginald Alexander Kidd, B.Sc. (*St. Andrews*).
Professor Harold Percy Philpot, B.Sc. (Eng.) (*Lond.*).
Matthew Tweedie Baillie Whitson, B.Sc. (*Edin.*).

And had admitted as

Students.

Charles Afford.
David Gemmell Allan.
William Thomas Frederick Austin.
John Harold Barnet.
Wilfrid Gordon Belcher.
David Young Borthwick.
Douglas Holland Bourne.
Brian Ingle Brough.
Richard Christie.
Stephen Michael Clarke.
John Richard Collins.
Cyril Donald Cooper.
Joseph Peter Corking.
James Reid Crichton.
Philip Morgan Crossley.
Charles Comrie Dewhurst.
Dermot John Dunn, B.A.I. (*Dublin*).
John David Eastabrook.
John Wilson Ewart.
Gordon Walter Fryer.
John Gault.
James Gilbert.
Derek Ernest Glover.
Alfred Charles Gunnee.
Gavin Clifford Hartley.
John Keith Maxwell Henry, B.A.I. (*Dublin*).
William Mackay Sinclair Houston, B.Sc. (*Glasgow*).
Philip Howarth.
Thomas Henderson Donald Hunter.
Sheikh Mohammed Ismail.
James Harvey Trevithick Ivory.
Joseph Ewart Bertram Jones.
John Simpson Keighley.
William Winston Lord.
Geoffrey Lowe.
Donald Murdo McCallum.
Donald Nicholson MacLeod.
Robert John Marshall.
Charles Frederick Martindale.
Donald Mills.
Linley Barrett Ollier.
Robert Albert Rayner.
Kenneth Rhodes, B.Sc. (*Leeds*).
Walter John Searle.
Mohan Lal Sharma.
Peter Allen Shepherd.
John Francis Stanbury.
Albert Edward Townsend.
Charles Thurston Walch.
Cecil Wallace Warne.
Henry Walters.
Lancelot Owen Vernon Watson.
James Stuart White.
George Francis Winters, B.E. (*National*).

The Scrutineers reported that the following had been duly elected as

Member.

Professor Emil Heinrich Probst, D.Eng. (*Charlottenburg*).

Associate Members.

Arthur Joseph Bishop, Stud. Inst. C.E.
Walter Rounsfell Brown, Jun., B.Sc. (*Edin.*), Stud. Inst. C.E.
William James Cameron, B.Sc. (*Glas.*), Stud. Inst. C.E.
Gordon Charles Coombe, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Henry Criswell, Jun., B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Robert Denis Fletcher, Stud. Inst. C.E.
James Fulton, B.Sc. (*Glas.*), Stud. Inst. C.E.
Ronald Gilbert, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
John Forbes Glennie, B.Sc. (*Bristol*), Stud. Inst. C.E.
Ronald Jack Goodwin, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Redvers William Grant, Stud. Inst. C.E.
Frank Herbert, B.Sc. (*Manchester*), Stud. Inst. C.E.
Homi Nusservanji Kanga, B.Sc. (*Edin.*), B.Sc. (*Bombay*), Stud. Inst. C.E.
Leslie Dopping Latham, B.A., B.A.I. (*Dubl.*), Stud. Inst. C.E.
Cuthbert Pullen Lewis, Stud. Inst. C.E.
Terence Harrison McGrath, B.Eng. (*Liverpool*), Stud. Inst. C.E.
George Leslie Moore, B.Sc. Tech. (*Manchester*), Stud. Inst. C.E.
James Melville Niven, Stud. Inst. C.E.
Stanley Norman Palmer, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Clifford Leonard Parkinson, Stud. Inst. C.E.
William Ernest Parsons, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
John Ashton Pool, Stud. Inst. C.E.
Hubert Woodroffe Potter.
David George Price, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Ralph Leslie Rolph, B.A. (*Cantab.*), Stud. Inst. C.E.
Norman Gerald Russell, B.Sc. (*Belfast*), Stud. Inst. C.E.
George Herbert Scott, B.Sc. (*St. Andrews*), Stud. Inst. C.E.
Frank Hutcheson Smith, B.Sc. (*Glas.*), Stud. Inst. C.E.
Daniel Sidney Sutton, Stud. Inst. C.E.
Sydney Kenneth Tolfree, B.Sc. Tech. (*Manchester*), Stud. Inst. C.E.
Arthur Walter Turner, Stud. Inst. C.E.
William Alexander Watson, Stud. Inst. C.E.
Edward Taylor Williams, Stud. Inst. C.E.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5216.

"The Sewage-Disposal of Delhi." †

By John Aldhelm Raikes Bromage, M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The Imperial Capital of New Delhi extends southwards from the city walls, whilst, away to the south-west, beyond the low range of hills known as the "Ridge," is the Military Cantonment. Each of the four areas (the Civil Lines, the City, New Delhi, and the Cantonment) has its own local administration. *Fig. 1* (p. 158) shows the general layout of the part of the area which is discussed in this Paper.

History.

The City of Delhi has been partially sewered for many years and the disposal of the sewage was by farming on an area to the south of the city walls. When plans for the construction of the Imperial Capital of New Delhi, immediately to the south of the city, were formulated, arrangements were provided whereby the whole of the city sewage was taken by large trunk sewers through the new area and was discharged into an outfall taking the sewage of both the old and the new towns. The sewage was pumped on to a sewage farm without any treatment except screening.

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† Correspondence on this Paper can be accepted until the 1st August, 1940, and will be published in the Institution Journal for October 1940.—Sec. Inst. C.E.

The scheme was designed to deal with a maximum sewage flow of 8,360,000 gallons daily (20 gallons per head for a population of 416,000) which flow, it was anticipated, would be reached by 1955.

The new capital has grown beyond all expectations and the population of the city has also increased. By 1933 the sewerage works were strained to their limit. It is estimated that by 1935 the population had reached 550,000

Fig. 1.

and that the dry-weather sewage flow was 14 million gallons daily. This original under-estimation had a far-reaching effect at the point Q (*Fig. 1*). At this point, all flow in excess of 3 times the dry-weather flow is discharged into a storm drain, the upper reaches of which are covered, the remainder passing down the outfall sewer to a pumping station at Kilokri. With the increasing use of water the sewage flow has risen correspondingly and, in 1936, averaged over 14,000,000 gallons daily at Kilokri pumping station, or 67 per cent. above the figure originally estimated for 1955. There are

definite "peaks" in the dry-weather-flow curve at the pumping station at midday and midnight, when the rate of flow reaches twice the average.

The original outfall sewer is a brick barrel, 66 inches in diameter, with a gradient of 1 in 3,000 and a flow capacity of approximately 36,000,000 gallons daily. This is 4 times the maximum flow for which the scheme was designed but only $2\frac{1}{2}$ times the average dry-weather flow of 1936. At times of rain, therefore, the sewer was unable to carry up to 3 times the dry-weather flow, with the result that foul water overflowed into the open storm-water drain at the point Q, thereby creating a nuisance.

Present Scheme.

A scheme for comprehensive extensions to the outfall and disposal arrangements was prepared in 1935 and received the sanction of the Government in 1936. The extended works are designed to deal with a dry-weather sewage flow of 24,000,000 gallons daily, with up to 3 times this amount at times of rain.

To achieve this end the outfall sewer from the point Q, which had a capacity of 36,000,000 gallons daily (*Figs. 2*, p. 160), was duplicated (*Figs. 3* and *4*, pp. 160, 161), whilst the screening plant was completely remodelled and hand cleaning of the screens eliminated. Detritus is removed from the sewage, after screening, by hydraulic plant, whilst the foul sewage runs on to the pumping plant.

Screening and Detritus-Handling Plant.

Original Plant.

The original outfall sewer, on reaching the Kilokri pumping station, bifurcated into two open channels, each 80 feet long and 8 feet wide. At the heads of these channels sloping bar screens were provided and these were cleaned by rakes moving on endless chains. Downstream of the screens the floor of the channel was hopper-shaped, and here detritus settled from the sewage, being subsequently removed by a bucket dredger to tip wagons at ground-level. The two detritus channels then united to form a single channel from which the main pump sumps were fed.

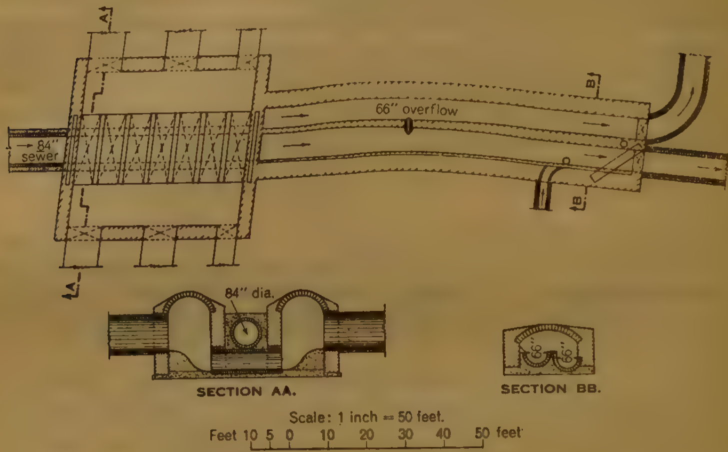
This section of the plant has not been entirely satisfactory and has been remodelled, its capacity being increased to deal with a 24,000,000-gallon daily dry-weather flow and 3 times that quantity at times of rain. The new layout is shown in Fig. 5, Plate 1. The whole extension was carried out without interruption of the sewage flow whilst much of the work had to be executed below water-level.

New Plant.

The new screens are located in a masonry well about 40 feet in diameter. Two screens have been installed, one on each side of the old sewer, whilst

in the space now occupied by the latter a third screen can be fitted if such ever becomes necessary. From each half of the penstock chamber the flow

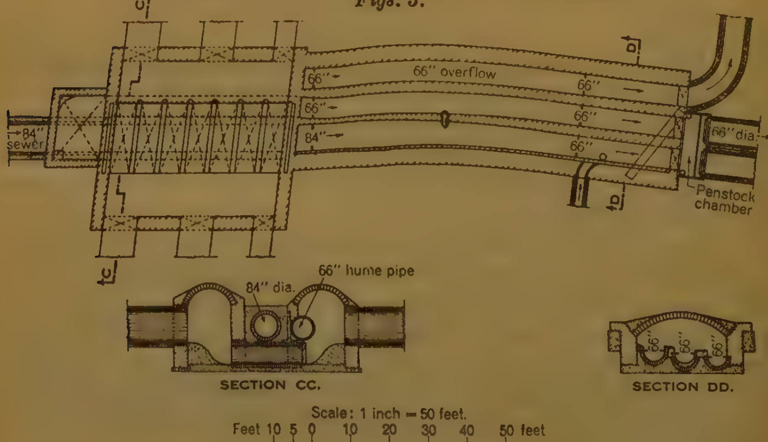
Figs. 2.



to one screen is controlled. The sewer inverts fall 18 inches when entering the chamber, thus giving a large screen area across the flow.

The screens, each 12 feet wide by 8 feet deep, are made up of vertical

Figs. 3.

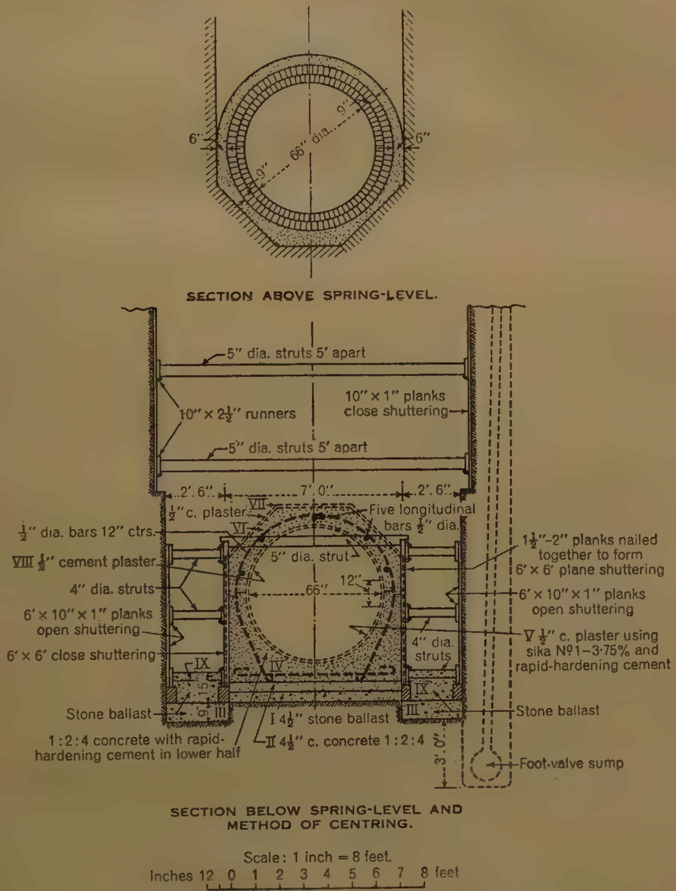


bars with $\frac{3}{4}$-inch clear spaces between the bars. The screens are cleaned by grab-type rakes. These rakes have no moving parts permanently submerged in the sewage and deliver the screenings on to an endless belt, which discharges them into tip wagons, whence they are removed to a trenching

ground. Hand-screening can be carried out, in an emergency, from a reinforced-concrete platform immediately above the screens. In India, the ignorance of the population makes the task of screening arduous because of the bulky material which is allowed to enter the sewers.

The screening chamber is covered by a reinforced-concrete operating

Figs. 4.



platform, whilst machinery at ground-level is covered by a superstructure which conforms architecturally with the adjacent pumping station. Incorporated in the superstructure buildings are the two detritus-pump wells and motor rooms, to which reference will be made later, a garage, an electricians' workshop, a smithy, and a small foundry. Ample illumination is provided above ground-level, whilst electric lights in bulk-head fittings on the underside of the floor-slab illuminate the screening chambers.

Separating Weir.

After passing the screens the sewage enters one of the two remodelled detritus channels, across each of which a masonry weir has been built. These weirs, which have curved upper faces, divert all normal flow, up to 47 cusecs total flow, into two ducts, one on each side of the plant, which communicate with the detritus tanks. Flow in excess of 47 cusecs passes over the weir and onwards direct to the pumps. Experiments showed that at times of flood the upper layers of the sewage contain little or no detritus and that there is no need to subject them to the detritus-removal process. Turbulence at the point of separation is avoided by a thin horizontal reinforced-concrete slab which acts as a cutwater, dividing the flow.

Over the masonry weir is a weighted tilting plate, so arranged that when sewage is passing over the weir there is always sufficient difference in head between the upstream and downstream faces to force a flow of the lower strata of the sewage, containing the heavier suspended matter, through the detritus tanks.

Detritus Tanks.

The detritus tanks are masonry wells 13 feet 6 inches internal diameter, sunk 39 feet below ground-level and plugged with concrete to form a hopper bottom. On entering a well the sewage is deflected downwards over a cone and then turns upwards through an angle of 180 degrees, during which movement road grit and other inert matter in suspension is deposited in the hopper bottom. The rising sewage is guided by thin radial walls in a streamline flow and runs into a number of reinforced-concrete troughs arranged radially around the well. From these troughs the sewage runs through a second reinforced-concrete duct and again enters the main channels downstream of the separating weirs. Penstocks in the ducts at the side of the main channels enable either of the detritus tanks to be shut off at will. The hopper bottoms of the old detritus channels have been filled with concrete and the channel floors now have even slopes to the pump sumps.

The grit deposited in the hopper bottom of the detritus tank is lifted in a water column by a "Karntclog" pump located in a dry-well immediately alongside the detritus tank. The mixture of water, grit, and organic matter (the latter in small quantities adhering to the grit) is delivered into a closed cylindrical settling-tank of steel carried over and athwart a tramway line. Baffles inside this chamber cause the heavy inert matter, washed clean of its adhering organic matter, to settle out into an air-lock, whence it is periodically removed into tip wagons. The water column from the pump, with any organic matter from the detritus, in suspension, passes on into the delivery pipeline of the main pumping plant. No head, and hence no power, is lost. Non-return and sluice valves control the flow towards the main pumps, whilst, by means of a by-pass at the non-return valve the whole plant can, if necessary, be flushed out by a

backflow from the main pumps. Such backflow would, for example, stir up any mixed deposit which might accumulate and partially solidify in the bottom of the detritus tank during a shut-down. Any bilge water collecting in the pump wells, the floors of which are below sewage-level, is removed by semi-rotary hand-pumps.

The motors operating the "Karntclog" pumps are, with their switchgear, located in buildings at ground-level. The pump well is covered by open-mesh steel flooring. The sewage channel from the screens and beyond the weirs to the suction sumps, is illuminated at night by a 1,000-watt floodlight.

Constructional Work.

The penstock chamber and the well which houses the new screens were first built round the old sewer without disturbing the latter or stopping the flow therein. This necessitated cutting under the old sewer in order that the floors of the new chambers could be built. The old sewer being a 66-inch diameter barrel in lime masonry of only 9-inch thickness, the undercutting could only be carried out in short lengths. The floors of the new chambers are about 5 feet below ground-water level, but the chambers are watertight.

In constructing each detritus well, excavation was carried down to within a few feet of standing-water level and the reinforced-concrete well curb was then cast in situ in a mould cut out of the hard soil. The well steining was then carried up to ground-level and sinking was effected by internal excavation. No superimposed load was required and electric pumps kept down the water during sinking. The pumps were in duplicate so as to ensure continuous running.

A layer of coarse stone ballast was first thrown into the well and into this ballast a 12-inch-diameter concrete pipe was placed vertically. Water was kept down by pumping from this pipe, the pump delivery being so throttled that water-level was kept just below the top surface of the ballast. Pressure from the sides of the well was relieved, during the construction of the plug, by making weep-holes in the steining and conducting the water entering through these along steel channels to the central pipe. Concrete was mixed at ground-level and was poured down timber shoots. Once started, concreting was carried on continuously.

Cast into the top of the concrete pipe was a short length of cast-iron pipe, flanged at its top end. After the concrete well-plug had been laid and set, the pumps were stopped, suction pipes removed and a blank flange bolted on to the top of the pipe. In this flange was a 2-inch diameter hole from which a pipe of the same diameter ran to ground-level. Through this pipe cement grout was poured which, running into the interstices between the stones under the plug, solidified the whole. The concrete pipe itself was filled with grout, the grouting being done under pressure. After the grout had set, the concrete pipe was cut off and the deflector in the bottom of the well was moulded.

The inlet pipe, also of concrete, was then cast, and next, the radial weirs, the final work being the casting of the guide vanes below the weirs which ensure a streamline flow within the tank. This last work had to be carried out in extremely confined spaces. During this period all weep-holes had been filled up and an absolutely watertight job obtained.

Operation.

A considerable amount of experimental work has been carried out on this plant with a view to obtaining the best operating procedure. The detritus of Delhi contains very fine dust of low specific gravity, and the object of the experiments has been to find an upward velocity through the detritus well which will allow the maximum quantity of detritus to settle without organic matter falling as well. This rate of flow can easily be varied by changing the weights on the hinged plates over the weirs in the sewage channels, and hence the head which drives the sewage through the well. An upward velocity of 1 foot per second gives the best results. Further experimental work has been carried out on the separator to ascertain the best rate of pumping to give optimum results. Orifices of various sizes were placed in the rising main from the " Karntclog " pump, whilst different forms of guide vanes and deflectors within the separator itself have at different times been fitted. A velocity of 1 foot per second through the rising main, and one-tenth of this in the separator body, which are represented by a discharge of 1 cusec from the pump, produced the best results, and the pump impellers have been finally designed to give this discharge. Charcoal, being refuse from domestic fires, is by far the largest constituent of the Delhi detritus.

PUMPING.

Capacity.

The maximum discharge to be handled by the enlarged pumping plant is 72,000,000 gallons daily. Two steam units out of the original 12,000,000-gallon sets of the old plant can run at a time, and provision has been made for installing six additional units in the new pumping station, either (*a*) all of 12,000,000-gallon capacity or (*b*) four units of 12,000,000- and two of 6,000,000-gallon daily capacity. As the maximum sewage flow will run for only a few hours in the year, the standby pumps provided are considered ample.

Power.

For the new units steam-, diesel-, and electrically-operated pumps were considered, whilst the advantages and disadvantages of generating electricity at the disposal works, using gas engines operating on sewage gas, were also considered. Delhi is 700 miles from the coalfields and 900 miles from the nearest port whence diesel oil is obtainable.

Preliminary investigations showed that diesel power would probably give the most economical results and would compete with electricity at 0·6*d*. per unit. A sludge-gas plant and gas engines with electric generators, on account of the high capital cost, gave an approximately equal figure. Steam, on account of the necessity of keeping a large portion of the boiler plant under steam ready for flows which may only come at rare intervals, was not an economical proposition.

The Delhi electricity authority then came forward with an offer to supply the new works with high-tension current (6,600 volts) at 0·6*d*. per unit with no standing kilowatt charge, and, after considerable discussion, this offer was accepted. The current is measured, for payment, at the receiving end of the line.

New Pumping Station.

The original suction channel, which served the old pumping plant, has been widened, and the new pumping station has been built on the opposite side of this channel. In addition to the new pumping plant with its switchgear, transformers, and auxiliaries, the new building houses a workshop, a large store, and administrative offices. The general layout is shown in Fig. 5, Plate 1.

Six penstocks, each 36 inches square and operated from a platform above highest sewage-level, control the flow from the feed channel to the six suction sumps. Each sump is 10 feet 6 inches square with its floor 3 feet below the channel floor. Considerable difficulty was encountered in constructing the sumps, as all are several feet below standing ground-water level. There was a constant inflow of water through the porous masonry of the old channel, in which the flow could not be stopped, even momentarily. Continuous pumping to keep the work dry was, therefore, necessary. Heavy timbering was required to support the existing works, whilst close timbering was used to support the wet sand and clay in which excavations were carried out.

Infiltration-water was taken along brick drains laid below concrete-level to a timbered well in No. 3 sump, whence it was lifted by centrifugal pumps. This well was finally closed by placing layers of graded stone around and over the pump suction, and over these a layer of sand. The concrete floor was then laid over the whole and around the pump suction, pumping being maintained until the whole pumping-station building had reached a considerable height above standing-water level. The suction pipe was then cut off and sealed, as already described in the case of the detritus wells, the whole then being finally covered by the sloping concrete forming the hopper bottom of the sump.

The suction sumps, as they were built up, were bonded in with the existing works. Leaks, where found, were grouted under pressure and the whole range of brickwork sump-wells was tested for watertightness before rendering.

The pumping-station floor is 20 feet below ground-level and 4 feet below standing-water level. Here the mass concrete was rendered water-tight by the addition of "Tricosal." The mortar of the lower parts of the brick walls was also waterproofed with the same compound, whilst a vertical damp-proof course on the outside of the wall extends to ground-level.

The excavation for the lower parts of the pumping station was carried out with sloping sides, whilst the deep walls are very much thicker at their bases than at ground-level (Fig. 6, Plate 1). This necessitated building portions of the superstructure on filled ground. In such places the walls are carried on reinforced-concrete beams running from the deep walls of the pump-room to solid ground outside the filled area. Floors over filled areas are reinforced-concrete slabs carried on these beams and on additional beams laid where required. A mezzanine floor built over the ground-floor offices and stores provides a large upper floor for additional stores.

In order to facilitate building work, the overhead crane in the pump-room was delivered 3 months in advance of the machinery, and, with its supporting rails and girders, was erected before the roof. A platform was constructed on the travelling girders and this gave invaluable assistance in the fixing and subsequent removal of the roof-slab shuttering.

Main Pumps.

Three new pumping units have been installed, and additional units will be purchased when required. The building has space for six units with all auxiliaries and switchgear. The pumping head of the new plant is the same as that of the old, namely, 56 feet maximum, the sewage being delivered to a well which forms the head of the duct running to the disposal works.

Each pumping unit consists of a horizontal split-casing pump with 18-inch diameter suction and 16-inch diameter delivery branches, cast integral with the lower half of the casing. The suction branch enlarges to 24 inches diameter, and, passing through the wall of the pumping station, terminates in a vertical bellmouth in the suction sump. Bronze impellers are fitted and the pump glands are sealed by filtered water under pressure. The capacity of each pump is 8,350 gallons per minute against the maximum working head of 56 feet, and 9,200 gallons per minute against a minimum working head of 47·75 feet.

Each pump is driven by a 210-brake-horsepower slip-ring induction-motor. Motor starters are of the totally-enclosed oil-immersed type, with special interlocking gear, to be described later. The motors operate on a 400-volt 3-phase 50-cycle supply.

Pump bearings are of the ring-oiled type, split on the horizontal centre-line so that the bearing bushes can be removed without disturbing the shaft alignment. A tilting-disk non-return valve is fitted on each pump delivery in addition to a sluice valve. In the base of all sluice-valve housings, except those on filtered-water pipe-lines, gunmetal scour plugs have been provided.

The three pumps deliver into a common 44-inch diameter rising main, into which the 20-inch diameter delivery from the detritus washing plant, already described, discharges. The rising main is carried 12 feet 11 inches above the pump floor on steel stanchions, thus permitting easy access to all parts of the plant. Space has been left for a second rising main into which the three additional units, to be installed later, will discharge.

Auxiliaries.

For priming the main pumps two electrically-operated exhausters are provided. The exhausted air is taken to a sewer vent-shaft running to the roof of the building. Glass tubes on the exhauster pipes, immediately above the pumps, indicate at once when all air has been removed. Liquid is thus kept out of the vacuum system. As sewage may, however, inadvertently get into the exhauster system, a filtered-water connexion is given thereto from the gland-sealing system to enable the air pipes to be flushed out through drains provided for the purpose.

For sealing the main-pump glands, filtered water is delivered to a small steel reservoir (800 gallons capacity) at ground-level. From this reservoir water is drawn by a single-stage electrically-operated centrifugal pump and is delivered, under pressure, to the main-pump glands. These pumps are in duplicate. An electrical interlocking gear is provided whereby the main pumps cannot be started unless one gland-sealing pump is running, whilst the latter cannot be started unless the reservoir, referred to above, is at least two-thirds full. Should the filtered-water reservoir become less than one-third full an alarm bell rings at the switchboard and a red lamp lights at pump-floor level. Gauges showing the gland-sealing water-pressure in each main pump are mounted alongside the suction and delivery gauges of these pumps.

Bilge water, from the glands and elsewhere, runs in an open floor drain to a collecting sump in the centre of the building. The normal flow into this sump is gland-leakage water and, being small, is removed periodically by a semi-rotary hand pump. Two electrically-operated bilge pumps are also provided for use in emergencies and these are connected by 3-inch diameter pipe-lines with the bottom of all suction sumps for de-watering the latter. As the contents of the bottom of these sumps will be of a semi-solid nature, open impeller-type pumps are provided, whilst a filtered-water connexion from the gland-sealing system is given to the bilge suction-piping to enable the latter to be flushed out. Bilge-pump glands are sealed in the same way as those of the main pumps, whilst they are primed, in the same way, by using the exhausters.

Steel stairways with open steel treads give access to the pump-floor and to the suction-sump operating gallery. A 7½-ton overhead travelling crane spans the pump-room, and at the south end a reinforced-concrete platform has been built at ground-level on to which laden trucks can be driven and so unloaded by the overhead crane.

Switchgear.

Outdoor air-break switches, fuses, and lightning arresters control the supply from the 6,600-volt overhead transmission-line from the power-house to the cable feeding the high-tension switchgear. The high-tension switchboard has three panels of draw-out switchgear, one for the incoming supply with integrating watt-hour meter and two outgoing panels for the two transformers.

Two 825-kilovolt-ampere oil-immersed indoor-type self-cooled transformers have been provided, and there is space in the transformer-room for a third. The normal ratio is 6,600–440 volts, with tappings for plus 2½ per cent., minus 5 per cent., and minus 7½ per cent. The transformers are mounted on wheels which run on steel joists embedded in, and flush with, the floor. These joists run the full width of the transformer-room and into the pump-room, thus enabling the transformers to be rolled under the overhead crane. A self-contained "stream-line" oil-filter of 15 gallons hourly capacity and mounted on rubber-tired wheels is provided for cleaning the transformer oil. This filter can be transported by motor lorry and will be used, also, for cleaning the oil of the transformers at the disposal works. The transformers are enclosed by an open-mesh metal screen. The equipment also includes a portable oil-insulation tester working at pressures of up to 50,000 volts.

The low-tension switchgear is mounted on three boards at ground-level in the pump-room. The main board contains seven panels of the draw-out type with oil-break switches. Two panels control the incoming supply and are provided with integrating watt-hour meters, voltmeters, and ammeters. Three outgoing panels control the supply to the three main pumps, whilst two more control the supply to the two sections of the detritus and screening plant. All panels are provided with red and green indicating lights. All outgoing supplies are separately metered. At the end of the busbars a four-core cable connects the main switchboard with the board controlling the supplies to the auxiliaries.

The auxiliary switchboard contains one incoming and nine outgoing panels, all with switches of the air-break type. The supplies to the exhausters, the gland-sealing pumps, and the bilge pumps, are each controlled by two panels, whilst one panel controls the supply to the workshop, another is for the lighting board, and another is spare, with plug fittings for connecting up portable apparatus. Here, as elsewhere, all outoing supplies are separately metered. The lighting board controls all lighting circuits on the whole estate. Circuits are across phases and are, as far as possible, balanced.

Performance.

The contract guarantees for the pump-plant and actual results obtained on the official tests are set out in Table I.

TABLE I.

	Guaranteed figures.	Actual figures (mean of three pumps).
Discharge at 56 feet head: gallons per minute	8,350	9,000
,, 47·75 ,, ,, ,,	9,200	—
,, 45 ,, ,, ,,	—	9,750
Overall efficiency of whole unit at 56 feet head: per cent.	70	77
Overall efficiency of whole unit at minimum head: per cent.	67·5	72·5
Temperature rise in windings on 6-hour test	95° F. (air-temperature 113° F.)	50·6° F. (air-temperature 70·2° F.)

Efficiencies of the three pumps did not differ by more than 2 per cent.

Rising Main.

The rising main is of cast iron to British Standard Specification, Class B. It is 44 inches in diameter and 700 feet long. A venturi-meter has been provided in the rising main, and the tube, in addition to the usual cleaning arrangements, is fitted with filtered-water flushing equipment. The meter panel is in the pumping station, adjacent to the main switch-board, and contains a rate-of-flow dial, flow integrator, flat chart recording flow and pressure, and a synchronous clock. The panel, which is wall mounted, is carried on hinges to facilitate access to the wiring.

The rising main terminates in a masonry well 13 feet 6 inches in diameter and this well is connected by a 44-inch diameter pipe with a similar well, nearby, at the head of the rising main from the steam plant. Both wells are covered and from the new well the gravity duct, 3½ miles long, runs to the disposal works.

Gravity Duct.

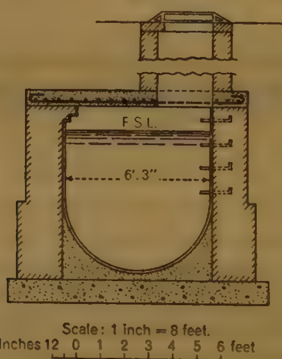
The gravity duct is of U-section, 7 feet deep and 6 feet 3 inches wide (*Fig. 7*, p. 170), and runs at an even gradient of 1 in 2,100 to the disposal works. The duct crosses over three large ravines, under many natural drainage lines, under two roads, and under one main line of railway. Several alternative routes were surveyed before a line giving an even gradient, yet clearing all obstacles, was found. At one point the cross-section alters, by easy slopes, to a flume 6 feet 3 inches wide and 5 feet deep, in order to cross under a drainage line; elsewhere, the section is uniform throughout.

For the first 3,000 feet the duct runs in the embankment which carried the old main distributary of the sewage farm. It then runs into a range of low hills and follows a fairly even contour along these to the disposal works, which are located on a spur from the parent range.

The duct is built of cement masonry with a cement-concrete invert. Where laid above ground and in cutting up to 3 feet of cover the roof is a reinforced-concrete slab. With cover greater than 3 feet, an arched roof is used. At the railway crossing the track is carried on a steel-girder culvert with only a few inches clearance over the duct roof, there being insufficient cushioning space for earth filling.

In the lowest section, including the railway crossing and one main-road crossing, deep rock cutting was necessary and special precautions, when blasting, were essential. The duct has been designed to carry 3 times

Fig. 7.



the ultimate dry-weather flow, or 72,000,000 gallons daily. The circular invert ensures good velocities at times of low discharge.

The construction of the first 3,000 feet of the duct necessitated closing the main channel feeding the old sewage farm, and, while this work was being carried out, about 9,000,000 gallons daily of crude sewage had to be run direct to the river after chlorination to the extent of 3 parts per million. The river at the time was in heavy flood, and ample dilution was obtainable. Manholes are provided every 300 feet and ventilation shafts every 600 feet.

Disposal Works.

Experimental Plant.

With a view to collecting data during the construction period, a small plant of 8,000 gallons daily capacity was constructed at the head of the rising main from the steam pumps and a portion of the sewage was turned through this plant. The periods of detention in the various sections of this installation and the form of the tanks were, essentially, the same as in the main plant. A temporary laboratory was also built and a considerable amount of valuable data collected.

Experiments on the irrigation of various crops using plain water,

effluent, and a mixture of the two, were carried out on virgin soil nearby, with the co-operation of the Government Agricultural Department. The effects of digging various proportions of mixed sludge into the soil were also investigated. India is primarily an agricultural country, and these experiments were initiated with a view to fostering the use of effluent and sludge for agricultural purposes. The poorest classes in India invariably dry the manure of their cattle for fuel, whilst imported artificial manures are beyond the means of the ordinary cultivator. If, therefore, prejudice against the use of sewage products as fertilizers can be overcome, the activated sludge of a purification works can be turned to valuable account.

Site of Disposal Works.

The disposal works are located 7¼ miles from the centre of Delhi on the Muttra road, near Okhla. The land, which was uncultivated and cheaply acquired, slopes away from the low hills along which the gravity duct runs, whilst from the effluent end of the works a large area, poorly cultivated due to lack of water, is commanded. Effluent can also gravitate to the Agra irrigation canal, whilst any surplus runs to the Jumna. In the rainy season, when irrigation requirements are nil, the river is in high flood and can take all the effluent, even if only partially treated.

The soil at the disposal-works site is a stiff *kankar* with an underlying stratum of rock. To enable the fullest use to be made of the rock, for foundations, a number of trial pits were excavated down to rock-level at points over the acquired area, and with the information gained therefrom a fairly accurate contoured plan of the underlying rock strata was obtained. The disposal plant was then located so that the deep tanks rested on, or were sunk a few feet in, solid rock. The rock is a mixture of blue quartzite and red *murram*. The former was used for concrete, but the latter is useless for any purpose.

"Simplex" Plant.

The "Simplex" plant, as at present built, has a capacity of 18,000,000 gallons daily dry-weather flow, and can pass 3 times this amount at times of rain. The plant is divided longitudinally into three sections and the works have been so completed that a further section, bringing the capacity up to 24,000,000 gallons daily, can be built. The general arrangement is shown in Fig. 8, Plate 1. Balancing channels between the different types of tank forming each section, and running the full width of the plant, enable any portion of the plant to be shut down without interfering with efficient operation.

The dividing walls between sections, and the southern external wall, which will, in due course, become a dividing wall, are designed to resist water-pressure, thus enabling any one section to be de-watered while other sections remain in operation.

The plant is made up of preliminary settling tanks, aeration tanks, and final settling tanks, having respective detention periods of 1½ hours', 12 hours', and 6 hours' dry-weather flow. The final settling tanks communicate with the effluent channel, whilst the activated sludge collected therein is pumped back again to the aeration tanks. Surplus activated sludge can be run into the incoming sewage, and, together with the heavier solid matter, is deposited in the preliminary settling tanks. The mixed sludge from the latter is run off, by gravity, to the sludge-drying beds, about ⅜ mile away. Alternatively, surplus activated sludge can be run direct into the main sludge draw-off channel.

Reinforced concrete has been used wherever possible, even for uptake tubes and for bridges, which are usually built of steel. In view of stone being available on and near to the site of the works, reinforced-concrete work could be carried out economically and future painting costs, which would have been incurred if steel had been used, have been saved.

Water.

At the disposal-works site, except for one small shallow roadside well, which dries up in the summer, there is no supply of water, and, before construction could start, a pipeline from the Delhi filtered-water system was laid from Kilokri to the new works. This line is 4 inches in diameter and will meet all requirements for operation and for domestic purposes. During the construction, however, the demand for water for concrete mixing, sand washing, and coolie camps was heavy, and was met by boosting the pressure at the head of the pipe-line during daylight hours. On completion of the work the booster pump was removed. Asbestos-cement pipes have been used on this main.

Influent and Feed Channels.

As it approaches the disposal plant, the gravity duct gradually changes section and terminates in a flume and broad-crested weir with a recorder for registering the flow. The recorder float is in a chamber alongside and is in hydraulic balance with the channel, but the chamber is arranged so as to keep out as much suspended solid matter as possible. The recorder has a local rate-of-flow indicator with which is incorporated electric transmitting gear, which, in turn, operates a rate-of-flow dial, an integrator, and a weekly rate-of-flow chart in the Assistant Superintendent's office on the works. Similar flumes and recorders are provided in the effluent and return-sludge channels, records of all these being kept on a three-panel board in the office.

The influent channel terminates in a feed channel running the full width of the plant. At the junction of these two channels, a portion of the surplus activated sludge and the whole of the liquor draining off the sludge-drying beds are turned back into the incoming sewage.

In the floor of the feed channel are twelve flat outlet-valves, each 24 inches in diameter, controlling the flow onwards to the purification plant. These valves are rubber faced and are operated by hand-wheels mounted on light bridges spanning the channel. The bearings of the operating spindles of these valves, and of all other valves and penstocks on the plant, are lubricated by grease-nipple and an electrically-operated portable "Tecalemit" grease-gun.

Preliminary Settling Tanks.

The twelve preliminary settling tanks are each 30 feet square, with vertical sides at the top and hopper bottoms. The tank pockets are constructed as reinforced-concrete frames resting on the solid natural soil and terminating in a cup-shaped base resting on or in solid rock. The panels of the frame are filled with cement concrete faced with bricks on edge, then rendered all over. All finished surfaces have been treated with three coats of silicate of soda. The tops of the "humps" between the pockets are of rubble stone masonry. Although water-pressure from behind the pocket walls is not expected, relief openings have been provided. These are 2-inch diameter holes filled with sand and gravel and covered with cement plaster. Should pressure from behind the slabs occur, the plaster will break and the sand and gravel will be washed out, giving a free passage for water. To have provided water-pressure resisting walls to all pockets would have involved unnecessary expense. The vertical walls of the tanks are of reinforced concrete and brick "sandwich" construction, with rubble masonry in the foundations to give additional stability. In so far as calculations for strength are concerned, the brickwork is not taken into account and acts only as shuttering for the concrete. With the difficulty of obtaining good joiners, experienced in shuttering work, and with bricks at 13*s.* 6*d.* per thousand, this method of construction has been found economical and satisfactory. The brickwork was laid in 1 : 2 cement mortar a few courses ahead of the concrete. Strict supervision and careful trowelling were necessary to obtain a good bond.

From the control valve in the feed channel, the sewage runs through cast-iron piping, 24 inches in diameter, to a deflector box in the centre of each tank. This box, made up of sheet metal, diverts the flow a few feet below the surface of the liquor in the tank, thereby preventing the disturbance of the top layers of the partially-clarified sewage.

A reinforced-concrete service bridge, 365 feet long and 3 feet 2 inches wide, extends over the whole range of these tanks, and from this bridge the deflector boxes are suspended. The weight of the cast-iron feed-pipe is taken by a reinforced-concrete stanchion built up from the framework of the tank pocket.

The heavier solids settle into the bottom of the hoppers and are drawn out from time to time under hydrostatic pressure, through 6-inch diameter de-sludging pipes, to a sludge channel incorporated in the dividing wall

between the preliminary settling and aeration tanks. The supernatant liquor runs over adjustable notched weir plates into steel troughs, four per tank, which communicate with a balancing channel, also incorporated in the dividing wall. The difference in level between the sludge channel and the overflow weirs provides the necessary head for forcing the sludge from the tanks to the sludge channel. Occasionally the sludge draw-off pipes choke and are cleaned by a high-pressure water jet. The necessary water-pressure is obtained from a centrifugal pump in an underground chamber at one end of the tanks, and a 3-inch diameter high-pressure main runs along the whole length of the tanks. Between alternate draw-off pipes, branches, wheel valves, and unions are provided to which flexible hose-pipes can be connected and passed down the blocked pipe. By this means a blocked draw-off pipe can be cleaned in less than 5 minutes.

The service bridge, with its supports, and the foundations for the latter, were the first parts to be built, and when these were completed the earth below was excavated. In this way the shuttering for the bridge concrete and the actual placing of the concrete were completed with a minimum of labour. The firm soil and a climate not subject to sudden changes permitted this procedure. Collapsible steel shuttering was used for the bridges and all the service bridges throughout the plant are of the same design, although, on account of varying stresses, the percentage of reinforcement in different spans varies. Details are shown in Figs. 9, Plate 1.

Aeration Tanks.

The aeration tanks are made up of one hundred and eight pockets, each 30 feet square, divided into three parallel units of thirty-six pockets each. The pockets are in rows of nine and cross baffle-walls are provided so that sewage, in passing through any section, follows a sinuous course. As it passes through these tanks a portion of the sewage is drawn through uptake tubes, in the centre of each pocket, by a revolving cone at the top, the latter throwing out the liquid so drawn up in a finely-divided spray. This causes the sewage to absorb atmospheric oxygen, and the finely-divided and colloidal matter in suspension, to form activated sludge.

The pressure-resisting walls between sections are made up of a vertical wall of reinforced concrete of "sandwich" construction, tied to a base consisting of two apron walls which also form the sloping sides of the adjacent tank pockets. The details are shown in Figs. 9, Plate 1. The long apron wall is used where the weight of the water, alone, resists overturning, whilst the shorter length is used at the cross-walls between pockets where there is the superimposed weight of the masonry of the hump to resist overturning. The boundary wall to the south, which, when the final extension is made, will become a pressure-retaining wall, is of the same construction. The walls have been tested under full water-pressure and are absolutely watertight.

The baffle-walls, which are not subject to unbalanced water-pressure, are made up of a reinforced-concrete framework of $4\frac{1}{2}$-inch square columns and a 15-inch by 6-inch reinforced coping with panels of $4\frac{1}{2}$-inch brickwork, cement plastered on both sides. The pocket floors are of concrete with a facing of $4\frac{1}{2}$ inches of brick, cement plastered. In these tanks, and elsewhere throughtout the plant, all sharp corners are rounded off, and all projections on which sludge or solid matter might lodge have been avoided.

Across the aeration tanks, 120 feet from the incoming end, is a bridge on which the cone driving motors are fixed. A service bridge, of the same design as that spanning the preliminary settling tanks, extends over the whole length of each row of nine aeration pockets. These latter bridges are in two sections, each section extending from one end of the tanks to the motor bridge. Both ends of each section rest on rollers, and hinged joints connect the supporting columns with the bridge itself. The bridge columns and their foundations, using wooden centring, were constructed first, holes just large enough for the purpose being excavated in the natural ground. The bridges were next completed, and afterwards the main tank excavation was carried out. The steel shuttering used for the bridges, although having high initial cost, enabled rapid progress to be made and resulted in an ultimate economy. By constructing the bridges before the excavation for the tanks was started, the height that the concrete had to be lifted, when placing, was kept to a minimum.

Over the centre of each pocket an opening was formed in the bridge floor, and here a cast-iron sole-plate was cast into the concrete. Each sole-plate forms the foundation for the gearbox and driving shaft of an aeration cone. The uptake tube in the centre of each pocket is of reinforced concrete, 3 feet in diameter, splaying out at the base to a skirt 6 feet in diameter. The tube is carried on four supports which leave a free space of 6 inches under the toe of the skirt. The reinforcement is made up of vertical bars and hoops, the former being carried out through the top of the tube and threaded to receive the nuts holding down the cast-iron angle-section ring which rests on the top of the tube.

The centering for the conical skirt was made of dry bricks, plastered, and, from the plinth so formed, collapsible wooden shuttering for the cylindrical portion was erected. The reinforcement was then placed in position and the whole was lined up with the opening in the bridge floor above; $\frac{1}{2}$-inch-mesh wire netting was wrapped around the outside of the reinforcement. Concrete was then placed inside the netting and was trowelled sufficiently to cause a rough plaster to form over the wires of the netting. The outer surface was rendered off smooth. The inside centering was then drawn out from the top, the sections being of such length as to admit of them being removed in the space remaining between the top of the tube and the underside of the bridge. The dry brick centering in the base was removed from above and below. After the removal of all centering the inside was rendered smooth.

The reinforced-concrete motor bridge, to which reference has already been made, is of ⊔ section and is made up of a number of double cantilevers, each cantilever being supported on four vertical columns at the point where the service bridges rest. Between the cantilevers, and resting on a horizontal expansion-joint, are suspended sections, making the whole into one long bridge. This arrangement ensures that no transverse stresses to the service bridges can occur, and hence no distorting stresses will be set up in the drive shafting and gearing.

Each cone-driving motor is of 36 horsepower and is of the totally-enclosed double-squirrel-cage type; it is suitable for working in the open. Over each motor, however, cowls are provided, these being painted aluminium colour so as to reflect as much heat as possible. The cables feeding the motors are laid on masonry brackets in the hollow portion of the motor bridge, and one cable feeds six motors. The cable trench is covered by removable reinforced-concrete slabs. Any rain-water collecting in the cable trench drains off, through suitable openings, into the aeration tanks. Mounted with each motor is a starter and ammeter. The motors drive the main shafting through "Morse" chains enclosed in dust-proof gearboxes with oil baths. A gauge outside each box shows the oil-level within. Arrangements for taking up chain slackness are provided. A short length of shaft and two bearings on the cantilever portion of the motor bridge takes the drive from the motor. At the junctions of the motor bridge with the service bridges, and between every pair of gearboxes on the service bridges, expansion couplings on the shafting are provided.

It may be noted here that the shade temperatures in Delhi vary from a degree or so below 32° F. to 118° F. On a full bridge length, therefore, the temperature movements are considerable. Ample provision has been made for this.

Aeration Cones and Gears.

Over the centre of each aeration pocket the driving shaft runs through a gearbox, provided with a clutch and bevel drive. The clutch enables any one bevel drive to be shut off without stopping the running of the shaft. The vertical shaft of the bevel drive operates the aeration cone. The base of the revolving cone comprises two concentric rings of cast iron and into the annular space between these fits the angle-iron, which forms the top of the uptake tube. The shafting and gearboxes having been aligned and bolted down to the soleplates, the ring on the top of the uptake tube was roughly placed in position and the cone attached to the bottom of the vertical shaft. The revolving cone and the stationary ring were then concentrically aligned, and the latter was bolted up and grouted into position.

The gears of the cone drive are of machine-cut case-hardened steel, and both the clutch and the bevel gears run in oil baths. All external bearings are lubricated by means of grease-nipples. Ball or roller bearings are used throughout. Should it be necessary to remove a complete gearbox

from a line of shafting, it can be replaced by a short length of shafting, of which a number of spare pieces, ready cut to size, with couplings, are maintained.

Service bridges are placed slightly off centre with respect to the aeration pockets. This results in the shafting being to one side of the bridge, which enables the operating staff to pass easily. All bridges are provided with handrails ; lifebelts are available at frequent intervals.

Draw-Off Arrangements.

At the end of the aeration tanks are twelve outlet troughs of steel, one trough to each row of cones. Each trough is suspended from two headstocks which enable the sill of the overflow weir to be varied over a range of 4 inches. This adjustment, in turn, varies the amount of submergence of the cones, and hence the amount of power consumed by them. Six outlet bends, each of 12 inches diameter, take the discharge from the weirs to a balancing channel between the aeration and final settling tanks. The vertical movement of the adjustable weirs is taken by rubber " concertina " rings between the overflow channels and the outlet bends.

Final Settling Tanks.

There are forty-eight of these tanks arranged in four rows. They are of the same design as the preliminary settling tanks and the layout is divided longitudinally into three sections. The mixed effluent and activated sludge from the aeration tanks pass from the balancing channel through twelve ranges of spun concrete pipes, each range having four 18-inch diameter outlets discharging through penstocks into deflector boxes, of the same design as in the preliminary settling tanks, over the centre of each tank. Reinforced-concrete service bridges, of the same design as elsewhere on the plant, span each row of twelve tanks, and from these the deflector boxes are suspended. The self-locking inlet penstocks, also, are operated from these service bridges. The reinforced-concrete feed-pipes are carried on reinforced-concrete bridges spanning the tanks.

The activated sludge settles in the bottom of these tanks, while the final effluent decants into forty-eight rows of collecting channels, each channel spanning four tanks. These channels have adjustable weir plates on their sides. As in the preliminary settling tanks, the effluent is received in a common collecting channel, incorporated in the eastern outer wall of the plant, and, after passing a recorder of design similar to that recording the incoming sewage, runs to the effluent channel communicating with the river.

Sludge Draw-Off Arrangements.

Automatic de-sludging gear has been provided in the final settling tanks. At the top of the sloping portion of each draw-off pipe is a cleaning eye, and

from these eyes the pipes run horizontally, being carried in groups of four across the settling tanks ; two groups of four (eight pipes in all) run to one de-sludging chamber. Each outlet is controlled by a hand-operated sluice valve and an automatic de-sludging valve. The latter are of the lifting type, and are operated through arms by cams on shafting which runs parallel with the outlet channels. At a point adjacent to this shaft, and midway between a pair of de-sludging chambers, is a further chamber containing a large float. This float, by means of a system of levers and a ratchet gear on the shaft, slowly turns the shaft as it rises and falls, and so operates the de-sludging valves. The float chamber is filled by a small pipe bringing effluent from the final settling tanks and is emptied by an automatic siphon when the water reaches a pre-determined level. By adjusting the sluice valve on this supply pipe the frequency of the cycle of float movements can be regulated. A further adjustment of the frequency of valve operation is provided by three sets of faces to every cam, which enable the opening period of any valve to be varied over wide limits.

Sludge-Return Pumps and Flume.

The activated sludge removed from the final settling tanks runs into a sludge draw-off channel with which the sludge-return pumps are connected. These pumps deliver the activated sludge into a reinforced-concrete flume, whence the sludge is returned back to the aeration tanks, the sludge channel, or the incoming sewage as required.

The four sludge pumps are of horizontal split-casing type with 15-inch diameter suction and 12-inch diameter delivery, each pump being capable of discharging 2,230 gallons of sludge per minute against a head of 15 feet. Pump glands are sealed with filtered water obtained by gravity from an overhead tank on the roof of the pump-house. Should the level of filtered water in this tank fall below a pre-determined level, a float-switch operates an alarm bell and light in the pump-room. Each pump is driven by a 20-horsepower slip-ring induction-motor. A 30-cwt. lifting block and traveller spans the pump-house.

The pumps deliver into an open channel forming part of the pumping-station building, and from this the sludge passes a recorder, of the same type as already described, and enters the box-shaped flume incorporated in the top of one of the pressure-resisting walls. Sludge is distributed from the flume through 18-inch diameter sluice valves to the three sections of the aeration tanks or elsewhere as required.

De-watering Arrangements.

The tank floors are all below ground-level, and, as gravity drainage would be far too costly, special de-watering arrangements are necessary to enable repairs to be carried out.

Within the wall between the preliminary-settling and aeration tanks a

9-inch diameter cast-iron pipe has been laid, with branches into each section of the aeration tanks, and a fourth branch, with blank flange, has been left for connecting up the last section of the plant, when built. Each branch is controlled by a sluice valve. All water in the aeration tanks above hump-level can be drawn out through this pipe by an 8-inch/6-inch centrifugal pump provided for the purpose. So as to avoid silting up, before the aeration tanks are emptied they would be filled with clear effluent by pumping water from the effluent channel into the upstream end of the aeration tanks. Pipe connexions in the underground chamber housing the pump, to which reference has already been made, are arranged so that one pump can do the two duties.

For emptying the deep settling tanks and the pockets of the aeration tanks a vertical-spindle "Karntclog" pump, suspended from the bridges, is used. Flexible delivery piping from this pump enables it to discharge where required. At convenient points on the service bridges, and around the plant, pillars with lighting and power plugs have been provided so that inspection lamps and the portable pump can be easily connected.

The whole plant is illuminated at night by four 1,000-watt floodlights, placed on poles at the corners of the installation.

Sludge-Disposal.

The sludge is dried on open beds about ¾ mile away from the purification plant.

From the sludge draw-off channel, between the preliminary-settling and the aeration tanks, a 12-inch diameter spun reinforced-concrete pipe runs to the drying beds. This pipe runs at an even gradient of 1 in 330, and a gravity discharge on to the top of the drying beds is possible as the ground slopes away at a steeper gradient than that of the sludge pipe. The site of the drying beds was selected with this end in view.

Climatic conditions in Delhi are favourable to sludge drying. The climate is dry for the greater part of the year, with an average annual rainfall of 40 inches. Nights in winter are cold and frosts are not uncommon, but high temperatures occur during the day. In summer, day shade temperatures reach 118° F., whilst night minima are frequently over 90° F. 90 per cent. of the rain falls in July, August, and September, but even then hot spells occur.

The drying beds are 790 feet long by 500 feet across, and are made up of 18 inches of graded gravel and sand. The beds are subdivided by five roadways into two end sections 75 feet wide and four other sections, each 150 feet wide. Each section is surrounded by masonry walls, on top of which run small reinforced-concrete sludge-distribution channels. From openings on these, controlled by shutters, the sludge runs on to the beds. Scouring of the beds is prevented by concrete slabs placed under each opening. Each section of the beds is again subdivided into three subsections. Most of the drying is by evaporation, but any liquor draining

through the beds is taken by channels in the concrete floor to a sludge-liquor channel at the lower end of the beds. From this channel the liquor is pumped back through a 7-inch diameter rising main, and is turned into the incoming sewage. Control of the sludge-liquor pumps is automatic, the pumps being started and stopped by switchgear operated by floats in the sludge-liquor channel. Should the level of the contents of the sludge-liquor channel rise beyond a pre-determined point, an alarm bell and light, operated by a float, come into operation. At times of heavy rain the water falling on to the large area of the drying beds is considerable, and arrangements have been made for taking excess liquid into a natural drainage.

A metalled road runs from the Delhi–Muttra trunk road to the drying beds, and entirely surrounds the latter. The roads which divide up the beds are of concrete and access to these is obtained from this circular road. Carts and motor trucks can, therefore, get close to the sludge and direct sales and removal are facilitated. Unsaleable sludge is removed in tip wagons, running on tramways laid along the concrete roads, and is dumped in an adjacent depression, acquired for the purpose, and which has storage for the whole output for many years. Sludge removal is done by hand and is carried on in the daytime only. Men are on duty at night to open the regulators, and the beds are illuminated by floodlights in the same way as the purification plant. Hydrants are provided along all the concrete roads to enable these to be washed down as required.

Breeding of flies gave unexpected trouble when the drying beds were first put into operation. Investigations proved that the eggs were not brought down in the sludge but were deposited by flies from outside. The problem has been to adopt a drying regime which results in the sludge being on the beds for a period less than the incubating period of a fly. This has been done, and sludge is turned on to the beds in 3-inch layers. The dumps are of considerable depth and little trouble is experienced from the dried sludge, the surface of which is consolidated by rammers. A certain amount of diffidence has been shown by cultivators to using the sludge, which has an unknown manurial value, but a lead in its use has been given in the Viceregal and other gardens in Delhi. It is probable that arrangments for digesting the sludge will be put into operation. Such arrangements were included in the scheme as originally projected, but were postponed on account of cost.

POWER-SUPPLY.

Electric Supply.

Electric current is brought from the New Delhi power-house, 7 miles away, through a 6,600-volt overhead transmission-line with two circuits. A branch from this line also supplies the pumping plant at Kilokri, a four-pole structure, with isolating switches, being provided at the junction point. The isolating switches are of the pole-operated outdoor type.

Similar switches are provided at the ends of the supply-line. A second overhead line, following a separate route, runs to Kilokri and provides an emergency supply should the main line break down.

Transformer and Switch-house.

The high-tension line terminates at the disposal works in the transformer-house, where the switchgear controlling the whole plant is located. This building is situated immediately at the end of the motor bridge, the most central point of control. The control-room measures 40 feet by 25 feet and is arranged for north-lighting only.

The high-tension switchgear comprises an incoming panel, with integrating watt-hour meter, and serves the three transformers, which are each of 525-kilovolt-ampere capacity. Two of these transformers will meet the ultimate maximum demand of the whole plant. The supply voltage is stepped down from 6,600 volts to 400 volts. $2\frac{1}{2}$ per cent. plus and 5 per cent. minus tappings are provided, together with a $7\frac{1}{2}$ per cent. minus tapping.

The main low-tension switchboard contains eleven panels. The first three control the incoming supply from the transformers; the next three control the outgoing supply to the cone-driving motors Nos. 1 to 6, 7 to 12, and 13 to 16 inclusive. The last group is not yet installed. Panel No. 7 controls the supply to the sludge pumps and chlorine-house, No. 8 to the plug points, and No. 9 to the sludge beds. Panel No. 10 is for lighting control and No. 11 is a spare. The first seven panels are of the oil-immersed type and the remainder have air-break switches. A subsidiary lighting-control board distributes the supply to four circuits supplying the transformer-house, the flood-lights, office and street lighting, and residential quarters respectively. Every outgoing supply is separately metered so that a complete analysis of costs is possible. A small workshop and a staff room with lockers and cycle-racks, are incorporated in the transformer-house building.

Chlorination.

Although, in the near future, most of the effluent will be sold to cultivators, it is necessary to be able to turn the whole flow at times, and a portion always, into the river Jumna. The point of discharge is about $1\frac{1}{2}$ mile downstream from the Okhla headworks of the Agra irrigation canal, and, at times, there is practically no flow in the river, although, on account of seepage, it rapidly recuperates. To avoid any risk to the riverside villagers a chlorination plant has been installed which is capable of delivering a dose of up to 3 parts per million to the effluent. A manometer-type chloronome has been installed and gas is supplied from 150-lb. cylinders, arranged in three batteries of six. Each group of six cylinders is carried on a weighbridge with self-indicating dial. The use of cylinders of larger

capacity was considered, but unloading facilities at Okhla station were found to be inadequate. Cylinders are carted from the railway by motor-truck and are unloaded and placed on the weighbridges or into store by an overhead traveller. When working at its full capacity of 600 lb. of chlorine gas per day, a supply of 30,000 gallons of water daily would be required for the absorption tower. Instead of filtered water, therefore, clear effluent is pumped from the channel for this purpose. The concentrated chlorine solution is distributed through a perforated ebonite diffuser in the channel bed.

Storm-Water.

Storm-water drainage from the new works has received careful consideration, and arrangements for this and for the siting and drainage of borrow-pits, where water might stagnate, have been carried out in close co-operation with the local anti-malarial control authority.

Office and Other Buildings.

To the north-west of the purification plant is high ground and on this the office, laboratory, and residential buildings have been built. The office building contains rooms for the Assistant Superintendent, clerks, and storage, and also a fully-equipped laboratory with balance-room, gas plant, refrigerator, and other equipment. This room is arranged with a long north-light plate-glass window.

Residential buildings include a bungalow for the Assistant Superintendent and quarters for the operating staff. All have electric light and independent water-flushed sanitation. Those persons who work during the daytime only are housed at Kilokri, where ample accommodation already exists.

Operating Results.

The purification plant was brought into partial operation in July 1938, and into full operation at the end of August. Official tests were carried out over an extended period in October. There was a complete failure of the rains in 1938 and the sewage, at the time of the test, was very strong.

The specification and guarantee provided that the plant should produce a non-putrescible effluent, having the minimum characteristics as set out below, when ascertained by standard methods of sewage analysis as laid down by the British Ministry of Health, from an average daily sample :—

(a) A biological oxygen demand, in 5 days, of 2·0 parts per 100,000.
(b) Albuminoid ammonia :—0·2 part per 100,000.
(c) Total suspended solids :—3·0 parts per 100,000.

The electric-power consumption of the purification plant, exclusive of lighting, as measured on the low-tension kilowatt-hour meters, was not to exceed 450 units per million gallons treated.

Test figures were :—

Biological oxygen demand:	parts per 100,000	maximum 1·2, minimum 0·74	mean 0·95.
Albuminoid ammonia :	parts per 100,000	maximum 0·14, minimum 0·12	mean 0·132.
Suspended solids :	parts per 100,000	maximum 1·30, minimum 0·60	mean 1·00.

The power-consumption when obtaining these figures was 430 units per million gallons.

Subsequent to the tests, efforts have been made to reduce the power-consumption: an effluent still complying with the specification has been obtained with a power-consumption of 370 units per million gallons over an extended period.

Sale of By-Products.

(*a*) *Effluent.*

The effluent channel, in passing to the river, crosses the Agra irrigation canal, and penstocks have been provided in the reinforced-concrete aqueduct whereby a portion of the effluent can be turned into the irrigation channel. As the illiterate population along the canal drink the crude water, sewage effluent to give a dilution of at least 1 in 150 is turned into the canal. Landowners along the effluent channel have become interested in the use of the effluent, and contracts for the sale of a portion thereof have already been made at rates 25 per cent. in excess of the irrigation canal rates. The demand for effluent has necessitated the construction of a new channel, running due south from the disposal works and parallel to the river. This channel will serve several thousand acres and it is unlikely that, after a few years, any effluent will normally reach the river. In the rainy season, when cultivators do not require water, the river is in high flood and a dilution of 1 in 1,000 or more is obtainable.

(*b*) *Sludge.*

Sludge sales are developing slowly, but large-scale demonstrations in its use have been carried out during the winter crop of 1938–39 and interest is growing.

Administration.

The Delhi Joint Water Board was constituted by an Act of the Central Legislature in 1925, and, by an amending Act in 1938, became the Delhi

Joint Water and Sewage Board. The operations of the Board's water department comprise the drawing of the water from the Jumna, its purification and its delivery to a series of reservoirs stretching about 9 miles along the Delhi "Ridge"; from these reservoirs, mains supplying the various local bodies take off, each being separately metered. Each local body controls its own distribution and its own sewers, these latter discharging into the trunk sewers of the Board. The Board controls the outfalls, the pumping, the purification, and the final disposal.

It is impossible, at present, to ascertain sewage-disposal operation costs, but for water the bulk-supply rate was 2·45*d.* in 1937–38 and 2·22*d.* in 1938–39, per 1000 gallons in each case. This charge covers all expenses, including administration, operation, repayment of loans, depreciation, provident fund contributions, and laboratory charges.

Contractors.

For the Sewerage Scheme, Messrs. Duncan Stratton and Company of Bombay were the general contractors for the whole disposal works and for the detritus and screening plant. Messrs. The Harland Engineering Company were the contractors for the Kilokri pumping plant. The overhead transmission-line and the rising main were constructed by direct labour, as also was the filtered water-supply. The remaining constructional works were constructed by local contractors of standing.

Cost.

The estimated cost of the whole scheme was £320,000 and the actual cost £293,000. The estimated period for construction was 24 months. Work commenced on the 1st December, 1936, and the scheme, complete in all respects, was put into operation on the 1st September, 1938.

Acknowledgements.

The Author was assisted during the construction period by Mr. S. M. B. Lal, B.Sc. (Eng.), Assoc. M. Inst. C.E., who was Resident Engineer on the works. The construction of the sewers was carried out by the Public Works Department.

The Paper is accompanied by eleven sheets of drawings, from some of which Plate 1 and the Figures in the text have been prepared.

Discussion.

The Author, in introducing his Paper, showed some lantern-slides of typical portions and stages of the work.

The President said that he had lived in Delhi as long ago as 1897, and the sewage-disposal then was carried out by means of carts which took the sewage to a trenching ground. He was not quite sure whether that system applied to the whole of the population of Delhi, which at that time consisted of nearly 200,000 people within a walled city ; there might have been some kind of water-borne sewage-disposal to an old type of sewage farm. It was something of a revelation to read in the Paper what engineers had now done to carry out the immense work of cleansing, a very vital matter in India, where the difficulties were much greater than most engineers in Great Britain had any experience of. The Author stated in his Paper that the day shade temperature in Delhi in the hot weather reached 118° F., but when he was in Delhi the temperature was sometimes as high as 127° F., and he remembered an occasion when for 10 days at a time the temperature at night did not fall below 100° F., while at the same time there was a very strong wind. When he said also that in the winter ice formed on pails of water in the garden, it would be realized what an enormous range of temperature had to be covered by the material and the design for work of the kind described in the Paper.

He thought the sewage-disposal works at Delhi were a very great credit indeed to those who had designed them and constructed them.

Mr. W. H. Morgan said that the Paper described a very interesting piece of main-drainage work, including sewage-purification plant, which would rank as one of the largest in the world. Of the plants which gave complete purification to the Royal Commission standard of purification, there were only three in Great Britain which served a larger population than that of Delhi: namely, the plants in West Middlesex, Birmingham, and Manchester. In fact, he thought that the plant at Delhi would be one of the twelve largest in the world.

With regard to growth of population, the Delhi experience was very similar to that of West Middlesex. In Delhi, the original estimate of 416,000 people, which it was anticipated would be reached in 1955, was exceeded by 30 per cent. at a date not less than 20 years earlier. In the case of West Middlesex, the Parliamentary estimate of 700,000 people, given in 1930 for the commencement of the scheme, was exceeded by 40 per cent., and the population became 1,000,000, whilst the figure of 1,200,000, which was anticipated and estimated for 1980, had already been exceeded. The growth of population in West Middlesex was, of course, phenomenal, but the growth in Delhi had also been very great.

With the growth in population, Delhi had also experienced an increasing use of water, which had risen from the original figure of 20 gallons per head per day to 25½ gallons per head per day in 1935. The British figures were usually between 30 and 40 gallons per head per day, and he had no doubt that Delhi would soon reach the 30-gallon mark.

The Author had designed his scheme for a dry-weather flow of 24,000,000 gallons per day, whilst purification plant had been put down to take 18,000,000 gallons per day, leaving room for extension. No estimate of the population to be served was given in the Paper, but, taking a basic figure of 30 gallons per head per day, he gathered that the plant could now serve about 600,000 people, and 800,000 ultimately. Would the Author enlarge a little upon that point ?

The Author had been fortunate in the availability and adaptability of his sites. The purification-plant site was conveniently located on the level, and was at a distance of about 7 miles from the centre of Delhi, and, as pointed out in the Paper, land had been acquired cheaply. What had been the cost per acre ? For the West Middlesex site £1,000 per acre had been paid. The foundations of the Delhi site were very good, and the Author had made the best use of them and had obtained from them materials for aggregates. The sludge-disposal site of the Delhi works was located about ⅝ mile from the purification works, at such a level that the sludge was delivered on to the drying area entirely by gravity discharge, thus doing away with all pumping ; that was a very great advantage.

The description given in the Paper was very largely of the constructional work, and contained many points of great interest. Every possible advantage had been taken of local conditions to obtain the most economical results and to overcome the difficulties with regard to supplies of material and skilled labour. For instance, the Author said in his Paper : " Reinforced concrete has been used wherever possible, even for uptake tubes and bridges, which are usually built of steel." The Author also used reinforced concrete and brick " sandwich " walls, owing to the difficulty of obtaining good joiners. He was amazed to find that the Author was able to obtain bricks at the price of 13*s*. 6*d*. per thousand, but he presumed that the quality of the bricks was poor, as the Author resorted to a certain amount of cement rendering. Cement rendering was not used in England if it could be avoided, owing to the subsequent peeling, and he wondered whether the Author had experienced any trouble in that respect due to the large range of temperature in Delhi.

The whole of the work at Delhi had been carried out in 1 year and 10 months, and that represented a speed which probably could not be improved upon in Great Britain. It was excellent, and did great credit to all those concerned.

As he had not visited India it was very difficult for him to compare the conditions there with those in Great Britain, but obviously materials and labour were very cheap in India, because the whole scheme was stated to

have cost £293,000, which was a remarkably low cost. If his estimate of a population of 600,000 were correct, the cost worked out at a little less than 10*s*. per head of population, whereas in Great Britain the figure would normally be not less than 30*s*. per head of population. Perhaps the Author would agree with him, however, if he said that more plant might be required for the different conditions in Great Britain. On the other hand, the cost of the water-supply, which was given in the Paper as about $2\frac{1}{4}d$. per 1,000 gallons, gave a very clear indication of the remarkably low figures which prevailed in the Indian public services. The Author stated that it was impossible, at the time of writing his Paper, to ascertain sewage-disposal operation costs. Had the Author since been able to ascertain anything about those costs, and could he give some information about them? It would also be of interest if the Author could give some details as to unit costs, which were always very useful, particularly in comparing such a scheme as that described in the Paper with schemes in other parts of the same country or in similar countries.

There was only one other small point that he wished to raise, and that was with regard to the sludge-drying beds. The Author said in his Paper that the breeding of flies gave unexpected trouble when the drying beds were first put into operation, but that that difficulty had been overcome by lifting the sludge within the incubation period of the fly. He was sorry to say, however, that he had not the slightest idea of the length of that period. Would the Author give that information?

Mr. R. G. Hetherington said that the Paper was of particular interest at the present time, when the tendency of sewage purification seemed to be more and more in the direction of the large district works. Mr. Morgan had referred to the West Middlesex, Birmingham, and Manchester schemes, and there were other such schemes, some of which had not yet been carried out.

It was stated in the Paper that it had been decided that the sewage-works at Delhi should not generate their own power by the use of methane, but should buy electricity from the public undertaking, and the electricity was purchased at the extremely satisfactory price of 0·6*d*. per unit. When he read the Paper he wondered whether that was the whole story about the use of sewage-gas or methane for generating power. After all, methane was not produced for the purpose of generating power; it was produced as a result of treating the sewage-sludge, and was a by-product from that. He noticed that in the latter part of the Paper the Author referred to the possibility of having to erect sludge-digestion plant in the future. If the Author did that, would he not have gone a good deal of the way towards spending the money which was necessary for the production of methane, and therefore a stage on the way to the generation of power? In other words, would the saving in capital cost which was effected by buying electricity be a permanent saving, and would there be no offsets against it in the matter of treatment?

He noticed that the whole of the surplus over 3 times the dry-weather

flow was discharged through storm-water drains to the river. That was a different practice from that which was followed in Great Britain, where the standard figure was 6 times the dry-weather flow. He also noticed that twice a day the average peak dry-weather flow rose to twice the average dry-weather flow; judging from experience in Great Britain, if the average rise were to twice, the rise would occasionally be to more than twice the normal dry-weather flow. That was to say, conditions would be getting rather near the point, even in dry weather, at which crude sewage would be discharged to the river. Consequently, of course, it followed that a comparatively little rain would cause an overflow to the stream. In India that might not matter, but it would matter in Great Britain; there there were slight showers of rain occasionally, whereas in India there appeared to be a very great deal of rain or none at all. He thought there would be great difficulty in Great Britain and a grave deterioration in the rivers if a standard of anything like 3 times the dry-weather flow were adopted as the setting for storm-overflows.

There was one statement in the Paper that appealed to him very much: that was that definite experiments had been carried out on the actual sewage that had to be dealt with, in order to determine the correct upward velocity to give the best results in the settling tanks. He ventured to hope that that was a practice that would be followed more often in the future. Those who were responsible for sewage works in Great Britain did not always consider the details of the actual sewage with which they had to deal, or take the trouble to ascertain the velocities which would produce the best effects.

The Author referred to encouraging the use of sewage sludge as a fertilizer, and was apparently finding in India what was found in Great Britain, namely, that a great deal of education was required to make the farmer appreciate the merits of sewage sludge as a fertilizer. There were two schools of thought in Great Britain: one said that the farmer did not appreciate the sewage sludge, and the other said that the farmer very soon found out that if he did not buy it nobody else would, and hence, as it had to be disposed of, he was not going to pay anything for it. He had always thought that the trouble with regard to the use of sewage sludge as a fertilizer in Great Britain was the fact that the farmer wanted a dry powder that he could store in a shed, and bring out and spread on the ground when he wanted to do so. It was a very expensive business to produce anything of that character in Great Britain, and it did not pay to do it. He knew of one case in which it was done, however, and he wondered whether or not, with the high temperatures that prevailed in India, it was possible to produce a sludge fertilizer which was a dry, or more or less dry, powder, which seemed to be what the farmer always required. The farmer disliked sludge which was brought to his farm in a tip-cart, and which was difficult to handle and had to be dealt with almost immediately.

Reference was made in the Paper to the heavy rainfall over the sludge areas, the rain having to be carried off in storm-water channels. The results of rain falling on sludge areas in Great Britain and being allowed to get away were rather dreaded, because the water was regarded as likely to be a very seriously polluted liquid, and the view was taken that surface water which had had any connexion with sludge beds ought not to be looked upon as surface water, but as foul liquid requiring treatment.

Mr. C. B. Townend observed that, as Mr. Morgan had said, no estimate was given in the Paper of the population to be served by the scheme. The capacity of the plant was given only as the volume to be treated. By a similar calculation, he had arrived at the same conclusion as Mr. Morgan, namely, that the scheme would ultimately cater for at least 800,000 people, and that the purification plant had already been constructed to serve from 600,000 to 700,000 people. On that basis, the works certainly ranked amongst the largest in the world, and of those which gave complete treatment to the standard mentioned in the Paper there were probably less than ten which served larger populations. Practically all of those outstanding plants had been built or reconstructed during the previous 15 years, and in every such case the activated-sludge process had been adopted. For those large activated-sludge plants, a universal preference had been given to the use of the compressed-air method. He thought the chief interest of the Delhi plant lay in the application of the alternative method of mechanical surface-aeration in a plant of such magnitude. In Great Britain, those who had to deal with sewage works were keenly interested in the comparison of the various types of plant which might prove to be best suited for particular local conditions, and they would be grateful if the Author would give them the reasons for the adoption of the "Simplex" method at Delhi. It was probable that the choice of plant had followed the submission of competitive designs by the various contractors interested in the supply of the equipment concerned; he understood that that was the customary practice in India. In that case, it was possible that the choice of plant would have been determined to some extent by the comparative ability of the contractor to use his knowledge of local conditions in producing the most economical design from the point of view of the constructional work, and that the same result might not be obtained if comparisons were made between the methods of compressed-air and mechanical surface-agitation by an independent designer. He would be glad if the Author would give some information on those points.

In Great Britain there had been many instances of experimental work employing those different methods being carried out side by side on the same sewage, and it would have been very interesting if the experimental plant which the Author mentioned had been able to supply comparisons on those lines.

From the results of the analyses of the effluent given at the end of the Paper, it could be seen that the plant was certainly doing very good work

with a quite moderate use of power. It was, however, very difficult to judge the performance of the plant without having the analysis of the incoming sewage on which the pollution-load could be assessed. The lay-out of the plant was extremely neat, and was a fine example of the manner in which modern methods had reduced the area of the site required to serve even the largest populations.

One was tempted to compare the design and capacities with English practice, but that, of course, was extremely difficult. The pollution-load per head of population was likely to be lower in India; temperatures were higher and favoured more rapid purification by the activated-sludge process and the quicker drying of the sludge. Flows were very much more uniform for at least 9 months in the year than in Great Britain, whilst in the rainy season the rivers were in high flood and the purification was not so important. That had a great influence on the design of tanks dealing with dry-weather flows, and avoided altogether the necessity for storm-water plant to deal with flows from 3 to 6 times the dry-weather flow, as was customary in Great Britain. All of those factors had been utilized to the full in the design of the Delhi plant, which, quite apart from low prices of materials and ingenuity of constructional work, was undoubtedly one of the most economical in existence.

The capacity of the sedimentation tanks certainly would be too small for conditions in Great Britain. The upward-flow rate, which worked out at 22 feet per hour at the daily peaks and 33 feet per hour at the maximum rates of flow handled, would pass forward a relatively high proportion of suspended matter to the aeration tanks, and that in turn would cause a large production of surplus activated sludge. In Delhi, where drying was so easy, no difficulty would be met with on that account. The capacity of the aeration tanks and the final settling tanks appeared to be more in keeping with English practice.

With regard to sludge-disposal, there was little doubt that the whole of the power requirements of the works and of the pumping station could have been supplied by the use of methane gas from digestion plant. At a place like Delhi, where alternative fuels were difficult to obtain, there would be something very attractive in making the works self-supporting in power requirements. With a supply of electricity at 0·6*d.* per unit and with no standing kilowatt charge, however, although a saving could no doubt have been expected by the use of methane gas from a digestion plant, it probably would not have been very large.

The other main advantages of sludge digestion in Great Britain, to which Mr. Hetherington had already referred, did not appear to apply with equal force in India. For instance, the conversion of the crude material to a product more readily dried was of little importance where natural drying was so rapid. Again, reduction in volume by digestion was not such a great advantage where labour was cheap and where fertilizers were needed. The fly-nuisance in dealing with undigested material appeared to

have been overcome quite successfully at Delhi. He therefore found it interesting to hear that, in spite of those facts, the adoption of sludge digestion was still under consideration. Could the Author give some additional information on that point?

With regard to the sludge-disposal works, the Author was particularly fortunate in having a site which was so favourable from the point of view of general situation and levels, even pumping having been eliminated. The extraordinary difference between drying conditions in Great Britain and in India was reflected in the very small area required in Delhi, even for undigested material. The allowance of 1 square yard to rather more than 15 persons was very small, and those responsible for sewage plants in Great Britain might well envy Indian conditions, which would overcome so many of their problems.

It was rather surprising that any difficulty should have been experienced in disposing of the dried product as a fertilizer. As far as could be judged, the final mixture should have a relatively high nitrogen-content, and in view of the heavy demand for fertilizers in India the material should be of great use for that purpose. More success appeared to have been achieved in disposing of the effluent, from which the city of Delhi was fortunate in obtaining some revenue. It was unquestionable that sewage purification in the future would be based more and more on the aim of complete disposal of all the end products, whether solids, liquids, or gases, and such waste products might well become the future sources of raw materials for industrial commodities.

In conclusion, he thought that the Author was to be congratulated on showing how all the benefits of modern science in the sphere of sanitation could be placed at the disposal of the Indian people by the organization of their own resources at a price which appeared to be within their capacity to pay.

Mr. G. Bransby Williams, speaking as one who had been dealing with the problems of sewage-disposal in India for more than 30 years, said that he had read the Paper with very great interest but also with a certain amount of disappointment. There was so much more that the Author might have stated about the Delhi sewage works. Sewage-disposal in India was in a very backward state, and very few towns had proper sewage-disposal works. When, therefore, works of the kind described in the Paper were constructed in India, it was very important, in his opinion, that the information given regarding them should be as complete as possible, as it might be a help in the designing of other works elsewhere. It was quite impossible for him in the time at his disposal to refer to all the points that had impressed him in the Paper, but there were a few matters to which he would refer and upon which he hoped the Author would give enlightenment in his reply.

With regard to the quantity of sewage that had to be dealt with, he would like to ask the Author for more precise information as to the population that was provided for in Old Delhi and in New Delhi, and the dry-

weather flow from each section. Another important point was the proportion of the water-supply that was expected to reach the sewers, which varied very much in different Indian towns. At Nagpur it was only 20 per cent. of the sewage, at Cawnpore it had increased from 50 per cent. to an estimated quantity of 80 per cent., and at Calcutta, and, he believed, also at Bombay, the quantity of sewage and the water-supply were approximately equal.

It did not appear to him that the quantity of sewage for which the works at Delhi had been designed could be considered to be extravagant. It had been suggested in the course of the Discussion that the dry-weather flow was intended to be 30 gallons per head per day. At Cawnpore a water-supply scheme was now being carried out to give 60 gallons per head per day to a population of 250,000 people, that was to say, a supply of 15,000,000 gallons per day, and it was contemplated that that would be increased in the not very distant future to 25,000,000 gallons per day. It was estimated that, when that scheme was in operation, the sewage dry-weather flow would be 48 gallons per head, and it was likely to increase still further. Having regard to the almost universal experience that in all large Indian towns the rate of consumption of water was rapidly increasing, a quantity of 30 gallons of sewage per head seemed to be rather small, and it was permissible to think that a miscalculation might have been made, and that in the not far distant future the works that were now being carried out would be found to be of insufficient capacity.

When the details of the scheme were examined, the position was rather obscure. The primary settling tanks were said to have a capacity of 1½ hours' flow and the aeration tanks one of 12 hours' flow. The actual capacity of those tanks, according to his calculation, was 875,000 gallons and 8,000,000 gallons respectively. At the figures stated, those were only sufficient for sewage-flows of 14,000,000 gallons per day and 16,000,000 gallons per day—that was to say, the quantity of dry-weather sewage that was flowing in 1936. Perhaps the Author in his reply would clarify the position by explaining exactly what quantity the initial scheme was intended to deal with.

As had already been suggested, it would be of interest to hear something more about the experimental plant, with the details of the experiments and results. There had been very little research carried out on sewage-disposal in India. Mr. Williams thought that he had been responsible for the only two large-scale experimental plants that had ever been constructed in India. One had been constructed for the Calcutta Sewage Disposal Committee in 1916, and the other had been constructed at Nagpur in 1928. A great deal of useful information had been obtained, but the science of sewage-disposal had advanced greatly in recent years. Some of the information obtained had penetrated into India and had been used in the design of recent sewage works, but it had not been generally realized that the data obtained from European sewage were quite inapplicable to

strong septic sewage in the Indian climate. More research, therefore, was necessary in India, and anything that the Author could add to what was already known on the subject would be very useful.

The problem of sewage-disposal in India was very difficult at any time, and it was made particularly so by the fact that an Indian municipality could afford at best to spend on its sewage works only a very small proportion of what would be considered necessary for a town of the same size in Great Britain. Generally speaking, he would say that an Indian town could afford to spend about one-third of what an English town would consider reasonable, and in those circumstances a good deal of makeshift work had to be done in India.

He thought he could suggest certain principles to which sewage-disposal works in India should conform. In the first place, the high standard of purity of effluents that was necessary in Great Britain was not required in India. In almost all cases a partial bio-aerobic treatment preceded by effective sedimentation would be adequate. Special attention should be paid to the clarification process, because the more effective that was the less work would have to be done in the aeration tanks and the smaller and the cheaper those tanks could be. In that connexion, he thought that some system of flocculation would be found extremely valuable. Further experiments were necessary on the type and size of tank that would be most suitable. Where there were activated-sludge works, the activated sludge would probably be a very efficient flocculant.

With regard to the processes of bio-aeration, which had already been referred to, the question was rather difficult, but he could only say that at the Nagpur experimental works it was found that the air-blowing system had advantages over the mechanical-agitation system. It was found difficult to keep the sludge in good condition in the latter system without considerably larger tankage than was required in the case of the air-blowing system. That conclusion seemed to be borne out at Delhi. For partial treatment with an activated-sludge plant, a tank-capacity of about 0·75 cubic foot per head should be quite sufficient in India, if the treatment were preceded by proper pre-treatment. At Delhi the capacity seemed to be about 4 times that amount.

There was generally no difficulty in India in disposing of sewage effluent on the land. There was usually either cultivated land or land that could be cultivated in the vicinity of the disposal works, and when the cultivators had overcome their prejudice to the sewage effluent—a prejudice which was very strong in the first instance but which soon vanished when the cultivators found how valuable the sewage effluent was—a demand for it sprang up, and the difficulty was then to prevent the people near the effluent channel putting unauthorized dams across it in order to divert the sewage on to their own plots.

He did not think that sludge digestion was the right system for India. The proper system there, he was quite certain, was to settle as much of the

sludge as possible and to compost it with the town refuse. The system of composting had its origin in India, and Indian conditions were particularly suitable for it. The production of a fertilizer was especially necessary in India, and the objections to the sludge, which had been referred to in the Discussion, were overcome by means of the compost.

The principles that he had suggested undoubtedly differed in some respects from those on which the Delhi works had been designed. He would be glad if the Author would state why the works at Delhi had been designed on the lines which had been adopted, and what alternatives had been considered. Incidentally, he might suggest that it seemed rather like putting the cart before the horse to design and construct works and then to experiment with the sewage. He thought that the sewage experiments might have taken place at an earlier stage.

He would like the Author to give some information as to the analysis of the crude sewage, for comparison with the effluent-analysis which was give on p. 183.

Finally, he would be glad to know whether the estimate of £320,000 represented the ultimate cost of the scheme or only the cost of the initial portion now being carried out, and whether the actual figure of £293,000 was for works identical with those for which the original estimate was made.

Mr. A. J. Martin said that the works at Delhi had been skilfully designed to meet the exacting requirements of the case, in view of the conditions in which they were constructed. Engineers in Great Britain could only sigh in vain for bricks suitable for the class of work in question at a price of 13*s*. 6*d*. per thousand.

The increase which had taken place in recent years, both in the population of Delhi and in the flow of sewage per head, was remarkable. The Author had stated that the sewage was formerly pumped on to the farm without any treatment except screening. Mr. Martin would be glad to know whether that statement held good down to the inception of the works described in the Paper, or whether there was any intermediate stage. If the present works immediately succeeded screening and land-treatment, Delhi was indeed fortunate, in that the design of the complete scheme had been postponed until the ultimate requirements had made themselves manifest. In Great Britain sewage works were usually laid down in so many stages, on one system after another, that Delhi was to be envied in being able to launch a complete fully-fledged scheme *de novo*.

He took it that there had never been any doubt that the sewage of Delhi would be purified by the activated-sludge process, but, like Mr. Townend, he would be interested to hear the Author's reasons for adopting the "Simplex" system rather than any of the other variants of the process. He (Mr. Martin) would also be interested to hear what proportion of sludge was retained in the aeration tanks.

The capacities of the aeration tanks and of the final settling tanks appeared to be somewhat large, especially in view of the high temperatures

experienced in Delhi. Mr. Martin could not help wondering whether those capacities had been adopted as an insurance against any further unexpected increase in the population and flow.

The Author had stated that the installation of digestion tanks was contemplated. As the liquid effluent was sold, would there not be also a demand for the liquid sludge? He believed that wet sludge was sold at Jamshedpur. If the sludge could be sold in a wet stage it could certainly be applied more readily to the land, and a great deal of expense in drying or digestion would be saved.

The Author referred in his Paper, in the section headed "Operating Results" (p. 182), to the "minimum characteristics" of the sewage effluent as ascertained by the standard methods of sewage analysis laid down by the British Ministry of Health. Should not that be "maximum characteristics"? The figures given for biological oxygen demand, albuminoid ammonia, and total suspended solids, were all maxima.

He did not think the Author stated in his Paper whether any nitrification took place. It would be interesting to hear whether any nitrates were found in the effluent.

Mr. C. D. C. Braine said that few who were interested in sewerage and sewage purification in the East could fail to be interested in the Paper, but when he began to examine the Paper in detail it was found to be lacking in essential information, and that, of course, detracted from its value to those who specialized in the subject. For instance, the original scheme allowed for an ultimate flow of 20 gallons per head per day, but by 1935 that figure had already reached 25½ gallons per head per day, and the scheme described in the Paper apparently allowed for a still greater rate, but, as previous speakers had pointed out, neither that rate nor the ultimate population were mentioned. Such information was vital for the right understanding of the work that had been done at Delhi. Mr. Williams had said that 30 gallons per head per day was not very much. That depended, he thought, on circumstances. In New Delhi all the houses presumably had an adequate supply of water, and, as he knew from his own experience, very large quantities of water might be used in such localities. When the old town was taken into account as well, the average might be quite a low figure, but the amount of water used by the ordinary European in a place like Delhi might be very large indeed.

The population of an Eastern town had very different habits from those of the population in Great Britain, and those habits markedly affected design. In Great Britain the peak flow of sewage occurred in the middle of the day, but in Delhi there were apparently two peaks, one at mid-day and the other at midnight, and he would like to know whether that applied to both winter and summer. He had found that there might be two peaks, one in the morning and one in the evening, but the midnight peak seemed unusual enough to call for an explanation. There was another point he might mention in that connexion: that was that in Great Britain, where

there was one peak, the strong sewage stayed in the aeration tanks most of the night, because it was only slowly displaced, and it therefore received a lengthy treatment, but where there were two peaks the duration of the sewage in the tanks was automatically reduced from 12 hours to something of the order of 6 hours. Therefore the work done in the aeration tanks at Delhi was really far better than appeared at first sight when reading the Paper.

In Delhi, as in most other Eastern towns, he imagined that quite a large proportion of the population preferred to use stones rather than toilet-paper, and those stones could be very troublesome in sewers unless velocities were kept high. At Delhi the velocity in the original outfall, which was laid on a gradient of 1 in 3,000, was only about 2 feet per second in dry weather (and, of course, dry weather prevailed during most of the year), which seemed somewhat low, and it would be of interest to know whether that sewer was really self-cleansing or whether special steps had not to be taken to remove debris from it. It would be noticed that the single large sewer downstream of the pumping station had a dry-weather-flow velocity of about 3 feet per second and a maximum velocity of nearly 4 feet per second.

Nothing was said in the Paper with regard to the construction of the sewers, and no longitudinal section was given, although two cross-sections of the 66-inch sewer were shown in *Figs. 4* (p. 161). The lower section was somewhat intriguing, because no mention was made of it in the text, yet such a form of construction would not have been adopted unless ground conditions were very bad indeed. No sewer costs were given, however, or any other information.

In referring to the screening plant, the Author mentioned difficulties caused by the ignorance of the population, but that comment would be more valuable if it were accompanied by figures giving the quantity of screenings handled per head of the present population served.

The detritus chambers appeared to be of the upward-flow Bochum type, and therefore were somewhat unusual, so that details of the tanks themselves and of their performance would be most interesting. No details, however, were given in the text or shown in the drawings. With regard to operation, an upward-flow velocity of 1 foot per second was mentioned, but that was no less than 5 times the upward-flow velocity in the grit-washing tanks at West Middlesex and other plants where the same principle was used, and it was a very much higher velocity than that used in the Bochum plant in Germany. Possibly a decimal point had been omitted, because to retain in the detritus tank the fine dust said to be found in the Delhi sewage he would expect an upward-flow velocity of about $\frac{1}{10}$ foot per second to be necessary; that, in fact, was the velocity stated to give good results in the separator, whatever that might be. Without quantitative figures, however, it was not possible to appreciate how successful the design was.

With regard to the purification plant, again essential details were lacking. For instance, not even an analysis of the crude sewage was given, and without that it was impossible to judge the plant in any way. The following example would show the need for an analysis. Taking two schemes on which he was working, both of which were for towns east of Suez, in what he would call Scheme A the water consumption was 18 gallons per head per day whilst in Scheme B it was nearer 400, but that would, of course, be cut down drastically. The two analyses, although typical, were admittedly snap samples and were expressed in parts per 100,000, and they represented opposite extremes. They were given in the following Table.

	Scheme A.	Scheme B.
Suspended solids	264	12·4
Oxygen absorbed in 4 hours	·107	4·02
Chlorine	20	13
Total organic nitrogen (N)	28·3	4·1

If the results at Delhi had been obtained from a sewage comparable to that of Scheme B they were not particularly noteworthy. On the other hand, they could not possibly have been produced by the plant provided if the Delhi sewage corresponded with that of Scheme A. Clearly, then, an analysis was essential information.

The maximum overflow rate for the preliminary settling tanks seemed to be about 5,000 gallons per square foot per day, which was somewhat high, even taking into consideration the long spells of dry weather. A high rate like that might be productive of large volumes of activated sludge.

With regard to the aeration plant, which was so important, discussion was rather difficult, because nothing was said about pre-aeration, which was now favoured by many, and nothing was said of re-aeration. No mention was made in the Paper of the optimum sludge concentration in the aeration tanks or of the quantities of sludge obtained, and the thorny questions of the successful disposal of the surplus activated sludge and the adequacy of the sludge-drying beds were not discussed at all. Enough was said, however, to show that sludge digestion should have been adopted in the first instance, and it was contemplated now. If sludge digestion were now necessary for the disposal of the sludge, it meant, of course, that power could be supplied at very little cost indeed, because it was a by-product of the process, but it would seem that that valuable by-product might be wasted now because a separate electric supply had already been installed.

He would have thought that, once the religious difficulties had been overcome, there would be no serious difficulty in the disposal of the sludge; he imagined that religion was the serious trouble. Where humus was so

scarce the sludge was very valuable. In South Africa there was not, as a rule, the sightest difficulty in disposing of it.

It appeared from the Paper that the sludge-drying beds had a concrete floor. Why was that thought necessary? It was bound to add considerably to drainage difficulties during the monsoon, and the water, very much befouled, as Mr. Hetherington had remarked, was not run back to the plant but was run on to the open countryside.

Although so much useful information was lacking in the Paper, it was clear from the first-rate effluent being discharged that not much was lacking in the works themselves. In fact, the results obtained were so good that they gave point to his belief that it was time that The Institution ceased to talk of "sewage-disposal" and instead used the term "sewage-purification." He always felt that "sewage-disposal" to-day was a misnomer. To the man in the street the term was associated with the gross smells and abuses of sewage treatment as practised a few generations ago, and so strong was the bias against it still that even now the public could hardly be persuaded that there was no serious fear of smells to-day. To his mind, the term "sewage-purification" not only left a different impression on the public, but it was a far more accurate description of what was actually being done, and from the professional point of view it sounded better. Consequently he seriously suggested that the word "disposal" should be used by The Institution only in connexion with land-treatment, disposal in water, or sludge-disposal, in order that The Institution might keep its nomenclature abreast of the times. He felt sure that such high-class work as had been done at Delhi should be referred to as "sewage-purification", as really befitted it.

Very little information could be derived from the costs given in the Paper; if they were itemized they would be much more interesting.

Mr. W. H. Hillier observed that the cost of the whole scheme was very low, and he thought it would be useful if the Author would give the cost of the purification plant alone. The power-consumption was also low, but without the strength of the sewage being given it lost much of its significance. He thought there was a tendency to attach too much importance to the power-consumption figures, because the debt charges on capital were almost invariably more than double the power cost, which was often less than one quarter of the total cost of purification. Could the Author give the total cost of treatment, including debt charges, wages, repairs, etc.?

Mr. Hillier wondered if the comparatively small capacity of the preliminary sedimentation tanks was due to the danger of septic conditions developing. If they had developed, had the Author considered using chlorination to prevent the septicity?

He would like to have some details of the preliminary-sedimentation-tank effluent, particularly in regard to suspended solids. The settling

tanks had small surface areas, which, he inferred, was due to the depth of the rock foundation limiting the surface area.

In his description of the action that took place in the aeration tanks, the Author had said: " As it passes through these tanks a portion of the sewage is drawn through uptake tubes, in the centre of each pocket, by a revolving cone at the top, the latter throwing out the liquid so drawn up in a finely-divided spray. This causes the sewage to absorb atmospheric oxygen, and the finely-divided and colloidal matter in suspension, to form activated sludge." He thought the way in which that sentence was framed was rather misleading. He would be inclined to say: " This causes the activated sludge which has been added to the sewage to absorb atmospheric oxygen, and to adsorb and oxidize the finely-divided and colloidal matter in suspension." Activated sludge was added to the sewage, and the essence of the activated sludge process, it seemed, was oxidation by the action of activated sludge in the presence of air.

The construction of the uptake tubes of the aeration tanks seemed very complicated. If the Author could have obtained pre-cast concrete tubes with spigot-and-socket or Ogee joints it would have very much simplified the construction.

The design of the detritus tanks was rather difficult to follow without a drawing, and he would be glad if the Author could give some drawings of those tanks. As Mr. Braine had said, the upward-flow velocity of 1 foot per second was very high, and it was surprising that dust and charcoal were expected to settle out in such tanks. Would the Author say whether the detritus tanks removed much detritus?

*** **Mr. A. P. Maddocks** desired to draw attention to the remarkable rate of increase in the population of Delhi—a phenomenon which had been noticed of late in other large towns of India, although not, perhaps, to so great an extent. Recent census figures were approximately as below:—

Year.	Population.	Decennial increase.	Percentage increase.
1901	208,000		
		28·000	13·5
1911	236,000		
		69·000	29·6
1921	305,000		
		135·000	44·3
1931	440,000		

In 1935 the population was stated by the Author to be estimated at 550,000, a 25 per cent. increase in 4 years. At that rate the increase in the decade would be approximately 75 per cent. and the total next year would be 770,000. The capacity of the new works would therefore soon become insufficient. It would be of interest to know whether any figures were available of the estimated population at the present time or of the

*** This contribution was submitted in writing.—Sec. Inst. C.E.

present average dry-weather flow of sewage. In 1933 Mr. Maddocks had had the honour to be asked to advise the Government of India with respect to the Kilokri sewage farm and the future arrangements for the disposal of the sewage of Delhi, and he had then visited the greatly overtaxed sewage farm. It was consequently of special interest to him to read the Author's account of the works which had since been carried out, which were bound to have resulted in a great improvement in the health and amenities of New Delhi.

The Author stated that a small experimental treatment-plant was constructed at Kilokri. Apparently that was a "Simplex" plant. Was the diffused-air method tried, or was that considered to be less suitable for Delhi sewage? It would be interesting to know the results obtained from the irrigation of various crops with effluent, with the nitrogen-contents of the effluent given to them. Valuable results would, Mr. Maddocks thought, be obtained if those experiments were continued with (*a*) diluted settled sewage, (*b*) partially-purified sewage, and (*c*) mixtures of liquid sludge and purified sewage effluent, of (in each case) varying nitrogen-contents up to a maximum of 1 part per 100,000.

In that connexion he desired to stress the Author's remarks with regard to the manurial value of the sewage. Most of the soil of India was in great need of both water and manure—the manure from the cattle being dried and used for fuel—and it consequently appeared highly desirable that during the greater part of the year, when the crops required both irrigation and fertilizers, no sewage, sewage effluent, or sludge should be discharged into a river to waste, if it were practicable to make them available to the cultivators. The value of the sewage nitrogen had been estimated at Rs. 2 (3*s*.) per head per annum, and he was informed that Chinese contractors paid the Shanghai Municipality at that rate for the Shanghai night soil. The value to the cultivators probably depended to some degree on the condition in which it was supplied, that was to say, on the extent to which the purification process had been carried. On the Poona sewage-effluent area, where sewage screened and passed through continuous-flow settling tanks of about 8 hours' capacity was diluted with canal water before being supplied to crops of sugar cane, 3 lb. of effluent nitrogen had been found to be equal in value to 2 lb. of nitrogen in artificial manures. Assuming that each person contributed 4½ lb. of nitrogen per annum, and taking an average price for artificial manures, the value of the sewage nitrogen worked out at approximately 2*s*. 6*d*. per head per annum. If the purification of this sewage had been carried a stage further, the nitrogen would probably have been readily assimilated by the crops and its manurial value would have been higher. Sugar cane could not take effluent all the year round, nor was the full value of the nitrogen given to the cane exacted from the cultivators, but they have paid as much as Rs. 180 (£13 10*s*.) for sewage effluent per acre of cane irrigated, and the revenue derived from the sale of effluents had been very considerable.

In the case of Delhi, with a present population probably approaching 700,000, at 2*s.* 6*d.* per head the value of the sewage nitrogen would amount to between £80,000 and £90,000 per annum. Whilst it was unlikely that anything approaching that sum would be obtained from the cultivators, in the course of time, as the value of the effluent and sludge was realized, if the best use were made of the nitrogen available the revenue might be expected to cover the cost of both pumping and purification, whilst the unrecovered value of the nitrogen could, and should, benefit the cultivators.

The level of the effluent channel, which permitted effluent to be discharged into the Agra canal or into an extended effluent channel constructed alongside the canal, afforded great flexibility in the manipulation of the disposal works, and the existence of an irrigation canal alongside the sewage-effluent channel was ideal for the economical treatment of the sewage and disposal of the products of such treatment, and made it possible to make the fullest use of the nitrogen-contents.

It would be of considerable interest if the Author would supply figures giving in parts per 100,000, and also in lb. per 1,000,000 gallons of dry-weather sewage, the average total nitrogen-contents of (*a*) screened sewage and (*b*) sewage effluent; and in lb. per 1,000,000 gallons of sewage treated, the average nitrogen-contents of (*a*) wet sludge and (*b*) dried sludge. Such figures would show the total amount of nitrogen available per 1,000,000 gallons of sewage in (*a*) effluent plus wet sludge and (*b*) effluent plus dried sludge, and the losses, if any, during the process of purification. (The sludge probably lost nitrogen during drying.)

A sewage effluent with as much as 0·9 part of nitrogen per 100,000 was probably suitable for most crops, and the Agra canal was available for dilution. Mr. Maddocks would, therefore, suggest that, to the extent to which there was a demand for sewage nitrogen, settled but unaerated sewage might be discharged into an extended effluent channel alongside the Agra canal, and diluted with canal water before being given to the cultivators. As the demand increased, the aeration tanks would be used to a reduced extent, with a considerable economy in working expenses and increased revenue from the sale of nitrogen. It might also be found practicable, if the sale of dried sludge were unsatisfactory, to discharge liquid sludge into this extended effluent channel along with the settled sewage, the amount of dilution by canal water being correspondingly increased.

In November 1933, as an alternative to the Okhla site, which, if found to be practicable, seemed likely to have marked advantages in certain directions, he suggested the investigation of a site for disposal works near Badarpur, about 4 miles farther south along the Muttra road. The considerably lower level of the land there appeared likely to make it practicable, if advantage were taken of the smaller velocities permissible in sewers of large diameter, to discharge the sewage into the disposal tanks by gravitation, whilst a considerable area of land could be irrigated without

pumping. In that case no pumping, of either sewage or effluent, would be necessary at any time, although it would probably prove to be desirable, when the value of settled sewage diluted with canal water, or of fully- or partially-purified effluent—after being tried on a considerable scale on the gravitation area—became appreciated, to provide low-lift pumps for one or other (or all) of those in order to command by means of a high-level effluent channel alongside the canal a sufficient area of cultivated land on which diluted settled sewage or effluent could be used. A possible objection to that site was the fact that, whilst the disposal works would still be in the Delhi area, the land that could be irrigated would mostly be in the area controlled by another Government (the Punjab). It was understood, however, that that was not considered an insuperable obstacle. It would be of interest to know whether or not Mr. Bromage had investigated the possibilities of that site, and, if so, with what result.

The Author, in reply, observed that the Paper, as originally submitted, was too long, and, among others, the sections dealing with the history of Delhi sewage-disposal and the investigations into the question of future population were omitted.

The scheme, or rather alternative schemes, prepared by the Author had been submitted to a detailed examination by a Government Committee on which all interests were represented, and the question of incidence of population was fully investigated. The flow of 24,000,000 gallons daily presumed a sewage flow of 32 gallons per head for a population of 750,000. With the removal of the Imperial Capital to Delhi, the increase in population was very rapid, but authorities were agreed that that rate of increase would tend to drop. The scheme, which had been financed with a 50-per-cent. Government grant and the balance on a 3-per-cent. basis, provided for the next 30 years. The present population was estimated at 550,000.

The old Kilokri sewage farm was used up to the inauguration of the new scheme. The farm had become such an intolerable nuisance, on account of smell, waterlogging, and mosquito-breeding, that all interested were agreed that in the new scheme irrigation without full preliminary treatment should not be adopted. The Royal Commission's standard of purity was accepted as desirable for the Imperial Capital, although possibly at other places such a high standard of purity might not be necessary.

The area southwards to Badarpur and several miles south thereof was surveyed, and by now, probably, a further irrigation channel was in operation, distributing sewage effluent to a large tract of country as far south as Badarpur. Levels did not, however, permit of a gravity flow from the disposal works far beyond that point, and disposal on to the land without any pumping would have been impossible as, at Kilokri, sewers were already below the highest flood-levels of the Jumna at Okhla.

Due to heavy irrigation, the water-table under the Kilokri sewage farm

had risen, and that rise had extended over an area far exceeding the limits of the farm. The lower reaches of the outfall sewers were thus several feet below water-level, and that accounted for the adoption of a somewhat unusual method of construction.

The seasons in India being sharply defined, rain occurred when the river was in heavy flood, and it was safe to discharge all flow in excess of 3 times dry-weather flow direct to the river. It had, in fact, been suggested that during the monsoon period it might be possible to discharge sewage without treatment direct to the river.

The gradients of the outfall sewers were flat, but without expensive reconstruction that could not now be avoided. In constructing the new outfall a slightly shorter route, and hence a better gradient, was obtained. Silting did occur, and the use of flushing-tanks and chlorination to avoid or reduce septicity had been considered. The sewage might be classed as a medium domestic sewage.

Experimental plants, both of the compressed-air and of the "Simplex" type, were installed, but, due to causes beyond the Author's control, the more elaborate work was only possible on the latter plant. Interesting results, using effluent, sludge, and added water on different crops planted in virgin ground, were obtained. With a root crop, for example, using a mixture of 50 per cent. water and effluent with added sludge at the rate of 5 lb. per square yard, a crop 3 times that when using water alone was obtained. Local cultivators were asked to view those experiments. Sludge was being used in the Government gardens and a lead in that connexion had been given by its use on the Viceregal estates.

The "Simplex" process was adopted on account of lower costs, whilst the mechanical plant used therewith could be readily understood and maintained by Indian labour.

Complete sludge digestion was recommended by the Author in his original scheme, but at the time it was considered too revolutionary for Indian conditions. As it was desirable to improve the load-factor of the Government power-station, the added advantage of cheap power arising as a by-product from the digestion process did not arise. On account of the larger volume of undigested sludge, the Author thought that digestion would, sooner or later, be adopted. It might be of interest to note that sludge digestion on a comparatively large scale had been in operation in Bombay, and from the Dadar sewage-disposal works at Bombay gas was being used for heating purposes in an adjacent hospital, whilst experiments were also being conducted using the gas, compressed in cylinders, for use on motor-lorries. Proposals for using the gas for vehicular propulsion were considered for Delhi.

The Author agreed that composting, as suggested, was an ideal process for India and would have been suitable for Delhi, but, unfortunately, all refuse-disposal arrangements were to the north of the city, whilst sewage-disposal was, naturally, to the south, on the downstream side.

Both the estimated and actual costs given in the Paper were for the scheme as constructed. The cost of the disposal works alone, with ancillary buildings, was £165,000. A few examples of unit costs might be given. Excavation up to 5 feet: 7*s*. 6*d*. per 1,000 cubic feet; brickwork: £3 to £4 10*s*. per 100 cubic feet; cement concrete (1 : 2 : 4 mix): £5 10*s*. to £6 per 100 cubic feet; labourer: 7*d*. to 9*d*. per day; bricklayer or carpenter: 2*s*. to 2*s*. 6*d*. per day. Bricks at 13*s*. 6*d*. per 1,000 were, of course, poor, by British Standards. Land for the disposal works was obtained at a cost of about £9 per acre.

The Author wished to amplify the description given in the Paper of the flow of sewage to the detritus wells. Flow up to 22 cusecs in each half of the plant (44 cusecs equalled approximately 24,000,000 gallons daily) was diverted by the masonry cutwater to the detritus well, the drop in water-level across the weir being sufficient to force the water through the well. On flows over average occurring, the water passed over the weir but was held up by the weighted plate until a slight rise in the surface-level caused sufficient pressure to lift the plate. There was thus always sufficient difference in surface-levels upstream and downstream of the weir to force the lower portions of the flow, containing the detritus, through the detritus well. The upward-flow velocity in this well rose to a maximum of 0·15 foot per second. The Author regretted that he had no records as to the quantities of detritus and screenings removed.

On the question of tank capacity, it was a matter for consideration whether a population or a gallon basis should be taken. The work done in an aeration tank was really a factor of the population, and if, for a given population, the flow be increased, then the weaker sewage would, in proportion, be easier to purify. The Author believed that there were two schools of thought in that matter.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October 1940; the Author, in his reply thereto, will deal further with certain points raised in the Discussion.—Sec. Inst. C.E.

BRITISH–AMERICAN ENGINEERING CONGRESS, 1939.

The following Paper was to have been presented at the British–American Engineering Congress at New York in September, 1939.

"The Crippling Load of a Compression-Member in a Framework with Stiff Joints."

By Professor CHARLES EDWARD INGLIS, O.B.E., M.A., LL.D., F.R.S., M. Inst. C.E.

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INTRODUCTION.

WHEN calculating the terminal bending moments induced in the members of a framework by the stiffness of the joints, it is customary to ignore the increased flexibility of a member which is acting as a strut, and its increased stiffness if it is acting as a tie, and this simplified theory at once leads to the conclusion that the process of superposition is applicable, and that the terminal bending moments will mount up and continue to mount up in direct proportion to the applied loads. Actually, this deduction is very far removed from the truth, and, if change of flexibility is taken into account, it will be found that terminal bending moments may in many cases diminish, and even change sign, as the loading increases, thus enabling a compression-member to withstand a much greater thrust than would be expected according to the ordinary theory.

Changes in flexibility may not affect stress calculations to any serious extent when the loading is quite moderate, but they may have a most

important influence on the behaviour of a compression-member which is stressed nearly up to the limit of failure, and, unless these changes are taken into account, it is impossible to predict with any real accuracy the ultimate load a given framework can withstand, or to form any reliable estimate of the margin of safety it possesses under normal working conditions.

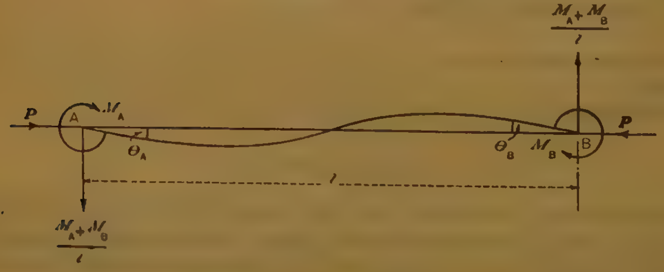
In the research which forms the subject of this Paper, the crippling load of a strut forming part of a framework with stiff joints has been investigated mathematically for a number of different cases, the analysis taking full account of the change in flexibility of members due to axial thrust or pull. Frameworks constructed to the dimensions taken in the calculations were then tested to ascertain how far the predictions based upon precise theory were confirmed or disproved by experiment, and in anticipation it may be mentioned that experiment was found to confirm theory to an extent which was quite remarkable.

General Theory.

Flexibility of a Member subjected to Terminal Bending Moments Combined with Axial Thrust.

Consider the member AB of uniform section loaded as shown in *Fig. 1* Taking A as the origin, y measured downwards, and x measured to the

Fig. 1.



right, the equation for the centre-line of the member is:

$$-EI\frac{d^2y}{dx^2} = Py + M_A - (M_A + M_B)\frac{x}{l}.$$

The complete solution of this is:

$$y = A\sin\frac{\alpha x}{l} + B\cos\frac{\alpha x}{l} - \frac{M_A}{P} + \frac{M_A + M_B}{P} \times \frac{x}{l}, \text{ where } \alpha^2 = \frac{Pl^2}{EI}.$$

Since $y = 0$ when $x = 0$, $B = \frac{M_A}{P}$; and since $y = 0$ when $x = l$,

$$A = \frac{M_A}{P}\tan\frac{\alpha}{2} - \frac{M_A + M_B}{P}\operatorname{cosec}\alpha.$$

Hence

$$y = \frac{M_A}{P}\left[\frac{\cos\alpha\left(\frac{x}{l} - \frac{1}{2}\right)}{\cos\frac{\alpha}{2}} - 1\right] + \frac{M_A + M_B}{P}\left[\frac{x}{l} - \frac{\sin\frac{\alpha x}{l}}{\sin\alpha}\right].$$

Since θ_A is the value of $\frac{dy}{dx}$ when $x = 0$,

$$\theta_A \frac{EI\alpha^2}{l} = M_A(1 - \alpha\cot\alpha) - M_B(\alpha\operatorname{cosec}\alpha - 1),$$

and similarly $\theta_B \frac{EI\alpha^2}{l} = M_B(1 - \alpha\cot\alpha) - M_A(\alpha\operatorname{cosec}\alpha - 1).$

From these two equations it follows that:

$$M_A = \frac{EI}{l\left(\frac{\tan\frac{\alpha}{2}}{\frac{\alpha}{2}} - 1\right)}[(1 - \alpha\cot\alpha)\theta_A + (\alpha\operatorname{cosec}\alpha - 1)\theta_B],$$

and $$M_B = \frac{EI}{l\left(\frac{\tan\frac{\alpha}{2}}{\frac{\alpha}{2}} - 1\right)}[(1 - \alpha\cot\alpha)\theta_B + (\alpha\operatorname{cosec}\alpha - 1)\theta_A].$$

For the symmetrical case where $M_A = -M_B = M$,

$$\theta_A = -\theta_B = \frac{Ml}{EI} \times \frac{\tan\frac{\alpha}{2}}{\alpha}.$$

For the particular case where $P = 0$, and consequently $\alpha = 0$, the expressions from M_A and M_B given above reduce to the familiar forms:

$$M_A = \frac{4EI}{l}\left[\theta_A + \frac{1}{2}\theta_B\right], \text{ and } M_B = \frac{4EI}{l}\left[\theta_B + \frac{1}{2}\theta_A\right].$$

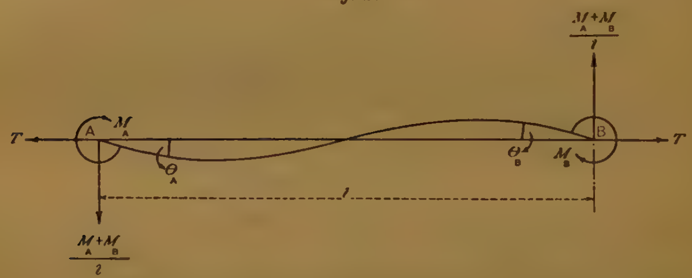
Flexibility of a Member Subjected to Terminal Bending Moments Combined with Axial Pull.

Consider the member AB, of uniform section, loaded as shown in *Fig. 2.*

Taking A as the origin, y measured downwards and x measured to the right, the equation for the centre-line of the member is :

$$-EI\frac{d^2y}{dx^2} = -Ty + M_A - \left(M_A + M_B\right)\frac{x}{l}.$$

Fig. 2.



The complete solution of this is :

$$y = A\sinh\frac{\beta x}{l} + B\cosh\frac{\beta x}{l} + \frac{M_A}{T} - \frac{M_A + M_B}{T} \times \frac{x}{l}, \text{ where } \beta^2 = \frac{Tl^2}{EI}.$$

Since $y = 0$ when $x = 0$, $B = -\frac{M_A}{T}$, and since $y = 0$ when $x = l$,

$$A = \frac{M_A}{T}\tanh\frac{\beta}{2} + \frac{M_A + M_B}{T}\operatorname{cosech}\beta.$$

Hence $$y = \frac{M_A}{T}\left[1 - \frac{\cosh\beta\left(\frac{x}{l} - \frac{1}{2}\right)}{\cosh\frac{\beta}{2}}\right] + \frac{M_A + M_B}{T}\left[\frac{\sinh\frac{\beta x}{l}}{\sinh\beta} - \frac{x}{l}\right].$$

Since θ_A is the value of $\frac{dy}{dx}$ when $x = 0$,

$$\theta_A\frac{EI\beta^2}{l} = M_A[\beta\coth\beta - 1] - M_B[1 - \beta\operatorname{cosech}\beta],$$

and similarly $$\theta_B\frac{EI\beta^2}{l} = M_B[\beta\coth\beta - 1] - M_A[1 - \beta\operatorname{cosech}\beta].$$

From these equations it follows that:

$$M_A = \frac{EI}{l\left(\dfrac{1-\tanh\frac{\beta}{2}}{\frac{\beta}{2}}\right)}[(\beta\coth\beta - 1)\theta_A + (1-\beta\,\text{cosech}\,\beta)\theta_B],$$

and
$$M_B = \frac{EI}{l\left(\dfrac{1-\tanh\frac{\beta}{2}}{\frac{\beta}{2}}\right)}[(\beta\coth\beta - 1)\theta_B + (1-\beta\,\text{cosech}\,\beta)\theta_A].$$

For the symmetrical case where $M_A = -M_B = M$,

$$\theta_A = -\theta_B = \frac{Ml}{EI} \times \frac{\tanh\frac{\beta}{2}}{\beta}.$$

For the particular case where $T = 0$, and consequently $\beta = 0$, the expressions for M_A and M_B given above reduce to the familiar forms:

$$M_A = \frac{4EI}{l}\left[\theta_A + \frac{1}{2}\theta_B\right], \text{ and } M_B = \frac{4EI}{l}\left[\theta_B + \frac{1}{2}\theta_A\right].$$

Terminal Rotation Produced by a Given Terminal Couple.

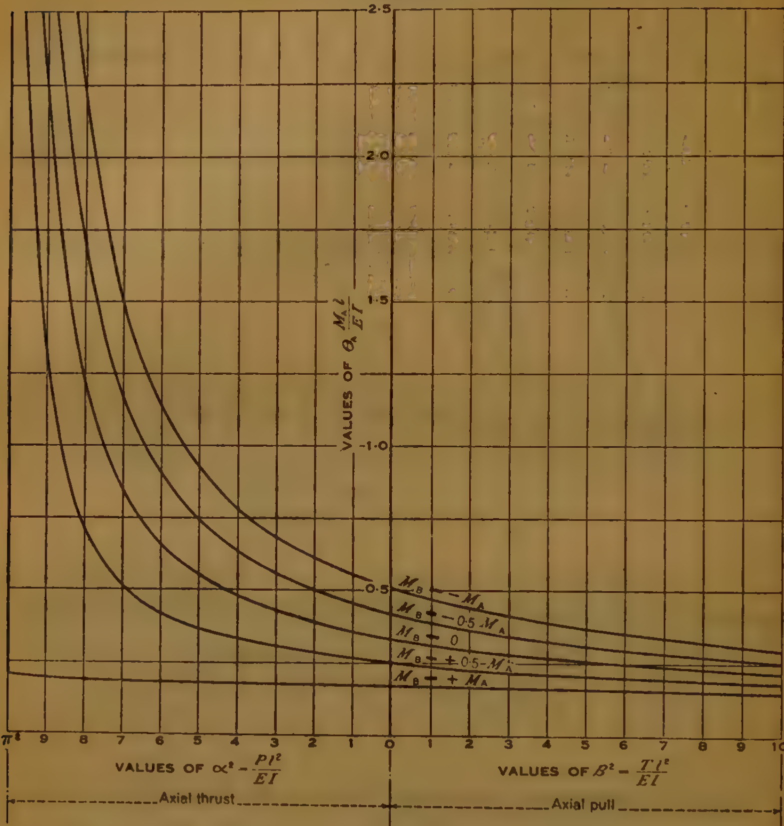
The manner in which the flexibility of a member depends upon its axial thrust or pull is made apparent by the graphs recorded in *Fig. 3* (p. 210), and, in computing secondary stresses in frameworks due to stiffness of joints, a theory which fails to take this change of flexibility into account can hardly be expected to yield reliable results.

For the case when $M_B = -M_A$, that is, when the terminal couples are equal in magnitude but opposed in direction, end rotation is seen to be particularly dependent on the magnitude of the axial load.

When $M_B = M_A$, and the terminal couples act in the same direction, the variation in flexibility is much less pronounced until the first Euler limit $\left(P = \frac{\pi^2 EI}{l^2}\right)$ is reached, when, as in all other cases, the resistance of the member to terminal couples falls to zero.

Writing
$$M_A = \frac{EI}{l}[A\theta_A + B\theta_B],$$

Fig. 3.



TERMINAL ROTATION PRODUCED BY A GIVEN TERMINAL COUPLE.

where $$A = \frac{1 - \alpha \cot \alpha}{\dfrac{\tan \frac{\alpha}{2}}{\frac{\alpha}{2}} - 1} \text{ for a strut, and } A = \frac{\beta \coth \beta - 1}{1 - \dfrac{\tanh \frac{\beta}{2}}{\frac{\beta}{2}}} \text{ for a tie,}$$

and where $$B = \frac{\alpha \operatorname{cosec} \alpha - 1}{\dfrac{\tan \frac{\alpha}{2}}{\frac{\alpha}{2}} - 1} \text{ for a strut, and } B = \frac{1 - \beta \operatorname{cosech} \beta}{1 - \dfrac{\tanh \frac{\beta}{2}}{\frac{\beta}{2}}} \text{ for a tie,}$$

the values of A and B for given values of α^2 and β^2 can be obtained from the curves set forth in *Fig. 4*.

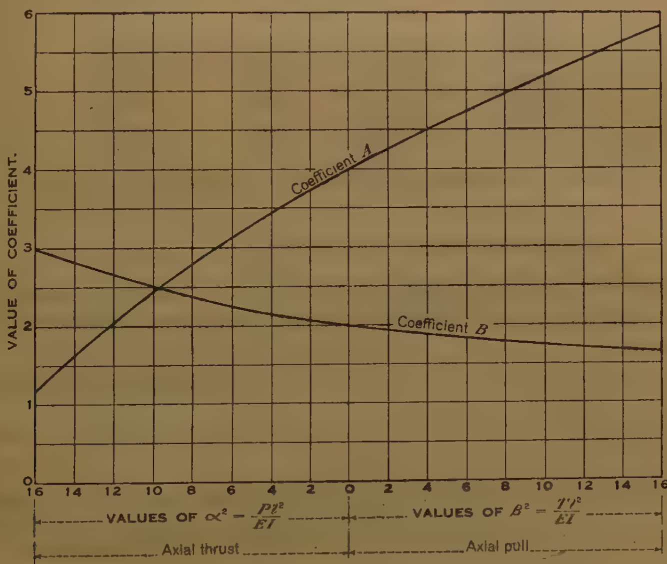
If the change in the flexibility of a member due to axial thrust or pull is ignored, the coefficients A and B will have the constant values 4 and 2 respectively, but the curves in *Fig. 4* show that this constancy in the values of A and B is very far removed from the truth.

The condition of equilibrium of a joint takes the form

$$\sum \frac{EI}{l}(A\theta_A + B\theta_B) = 0,$$

the summation including all the members radiating from the joint. The values of A and B in terms of α^2 and β^2 are set forth in Table I (p 212). From this Table or from the curves of *Fig. 4*, the values of A and B which

Fig. 4.



are appropriate to a particular member and to the load it sustains can be read off, and the equations of equilibrium are no more complicated and no more difficult to formulate than the equations of the usual type

$$\sum \frac{EI}{l}(4\theta_A + 2\theta_B) \doteq 0.$$

For the safe limits of loading usually adopted, the ranges of α^2 and β^2 will seldom exceed 5 ; but in considering the maximum loading a structure can support, values of α^2 and β^2 almost up to 16 may have to be taken into account.

TABLE I.—VALUES OF A AND B REQUIRED FOR THE FORMULA

$$M_A = \frac{EI}{l}[A\theta_A + B\theta_B],$$

when $$\alpha^2 = \frac{Pl^2}{EI} \text{ for a strut, and } \beta^2 = \frac{Tl^2}{EI} \text{ for a tie.}$$

α^2	A	B	β^2	A	B
0	4·000	2·000	0	4·000	2·000
1	3·865	2·035	1	4·131	1·968
2	3·726	2·071	2	4·259	1·937
3	3·583	2·110	3	4·385	1·908
4	3·436	2·152	4	4·507	1·881
5	3·284	2·197	5	4·627	1·856
6	3·128	2·245	6	4·744	1·832
7	2·966	2·296	7	4·859	1·809
8	2·798	2·352	8	4·971	1·787
9	2·624	2·412	9	5·081	1·766
10	2·443	2·476	10	5·189	1·747
11	2·255	2·546	11	5·294	1·728
12	2·059	2·622	12	5·397	1·711
13	1·853	2·705	13	5·500	1·694
14	1·638	2·796	14	5·601	1·678
15	1·412	2·895	15	5·700	1·662
16	1·173	3·004	16	5·797	1·648

EXPERIMENTAL VERIFICATION OF THEORY.

To test the validity of the foregoing general theory, analytical calculations relating to a framework with stiff joints were checked by direct experiment.

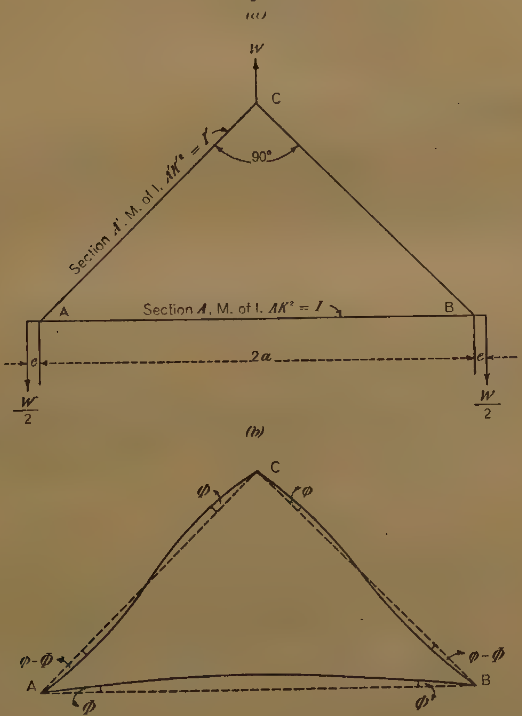
The general form of the framework and the nature of the loadings are indicated by *Figs.* 5 (*a*). The thrust in the member AB is $\frac{W}{2}$, the pull in the members AC and BC is $\frac{W}{\sqrt{2}}$, and ϕ, the increase in the angles at A and B which has to be resisted by the stiffness of the joints, is given by :

$$\phi = \frac{W}{2E}\left[\frac{1}{A} + \frac{\sqrt{2}}{A'}\right].$$

If Φ denotes the clockwise and counter-clockwise rotations of the joints A and B, the terminal angular deflections for the various members are indicated in *Figs.* 5 (*b*). By altering the eccentricity e shown in *Fig.* 5 (*a*), the rotations of the joints A and B can be increased or decreased, and in this way the behaviour of the member AB under the action of terminal couples of different intensities was studied. By employing frameworks of different sizes, the effect of the slenderness ratio $\frac{l}{K}$ on the crippling load of the member AB was also investigated.

If M_A and M_A' denote the terminal couples at A for the members AB and AC respectively, the exact theory which takes into account the increased flexibility of AB due to axial thrust and the increased stiffness

Figs. 5.



of AC and BC due to axial pull, gives

$$M_A = \frac{EI}{2a} \times \frac{\alpha}{\tan\frac{\alpha}{2}} \times \Phi, \text{ where } \alpha^2 = \frac{4a^2}{K^2} \times \frac{W}{2EA},$$

and

$$M_A' = \frac{EI'}{a\sqrt{2}}\left[\frac{\beta\coth\beta - 1}{1 - \dfrac{\tanh\frac{\beta}{2}}{\frac{\beta}{2}}}(\phi - \Phi) + \frac{1 - \beta\,\mathrm{cosech}\,\beta}{1 - \dfrac{\tanh\frac{\beta}{2}}{\frac{\beta}{2}}}\right],$$

where

$$\beta^2 = \frac{2a^2}{K'^2} \times \frac{W}{\sqrt{2}EA'}.$$

For equilibrium of the joint A, $M_A = M_A' + \frac{1}{2}We$. Hence the value of Φ

for any given load can be obtained, and the corresponding value of M_A then determined.

The centre-line of AB is given by the equation :

$$y = \frac{2M_A}{W}\left[\frac{\cos\dfrac{\alpha(x-a)}{2a}}{\cos\dfrac{\alpha}{2}} - 1\right].$$

The central deflection has the value

$$\frac{2M_A}{W}\left[\sec\frac{\alpha}{2} - 1\right],$$

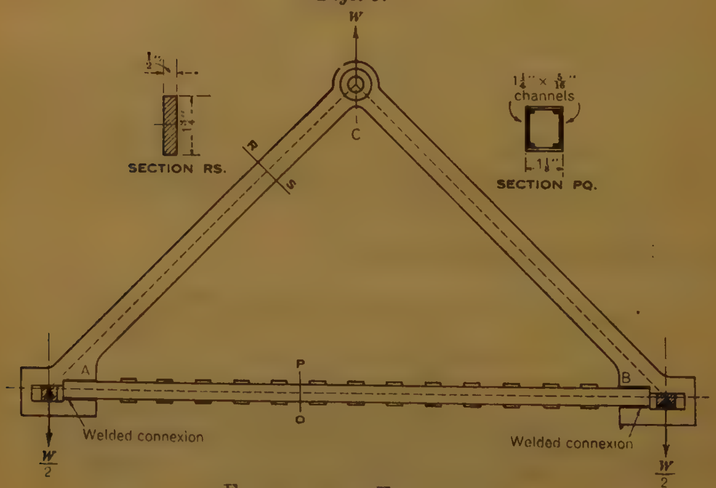
and the central bending moment is

$$M_A \sec\frac{\alpha}{2}.$$

TESTS.

Figs. 6 give details of the construction of a framework of the type described. The L-shaped portion ACB was cut out of a mild-steel plate

Figs. 6.



EXPERIMENTAL FRAMEWORK.

$\frac{1}{2}$ inch thick and the member AB was formed of two $1\frac{1}{4}$-inch by $\frac{5}{16}$-inch channels braced together by batten-plates. The stiff-jointed triangle was achieved by welding the ends of AB to the L-shaped portion ACB. The load was applied in a tensile testing machine, through a shackle at C, and resisted by shackles at A and B bearing on knife-edges. These knife-

edges, mounted in blocks, could slide in slots machined in the framework, and in this way any desired eccentricity in the loading at A and B could be achieved.

The central deflection of the member AB was measured by a gauge fitted to a steel bar, which rested upon pins projecting from points vertically above the centres of the joints at A and B.

Five different frameworks were constructed and tested, in which $\frac{l}{K}$ for the member AB had the values 60, 80, 100, 120, and 140, the framework in each case being an isosceles right-angled triangle.

For each framework five different values of the eccentricity e were taken, corresponding to the cases where the ratios of the secondary to the primary stress in AB as calculated by the ordinary "constant flexibility" theory had the values 0, 10 per cent., 20 per cent., 30 per cent., and 40 per cent.

The sections of the members were the same in all five frameworks, and the particulars of these sections are as follows:—

$$A = 0{\cdot}610 \text{ square inch}, \quad I = 0{\cdot}1011 \text{ inch}^4 \text{ unit}, \quad K = 0{\cdot}407 \text{ inch}.$$
$$A' = 0{\cdot}875 \text{ square inch}, \quad I' = 0{\cdot}2233 \text{ inch}^4 \text{ unit}, \quad K' = 0{\cdot}505 \text{ inch}.$$

As a sample of the calculations and experiments, the Author will consider the case where $\frac{l}{K}$ for the member AB had the value 120. For this case $l = 48{\cdot}84$ inches and $a = 24{\cdot}42$ inches.

If e is made 0·264 inch, the ratio of secondary to primary stress in the member AB for small values of W is 20 per cent. The theoretical calculations appertaining to this case are set forth in Table II (p. 216).

In *Fig. 7* (p. 217) the values of the terminal and central bending moments for the member AB are plotted in terms of its axial thrust, and it should be noted that, whereas the central bending moment mounts up with increased rapidity as the axial thrust increases, the terminal bending moment, after reaching an early maximum, decreases to zero and changes its sign when the axial thrust reaches the first Euler limit, which in this case has the value of 5·65 tons. For axial loads beyond this limit the reversed terminal bending moments produce points of contraflexure in the member, as indicated by the points P and Q in *Fig. 8* (p. 217), and thus, by partially neutralizing the bending moments due to axial thrust, they may be regarded as beneficial in character.

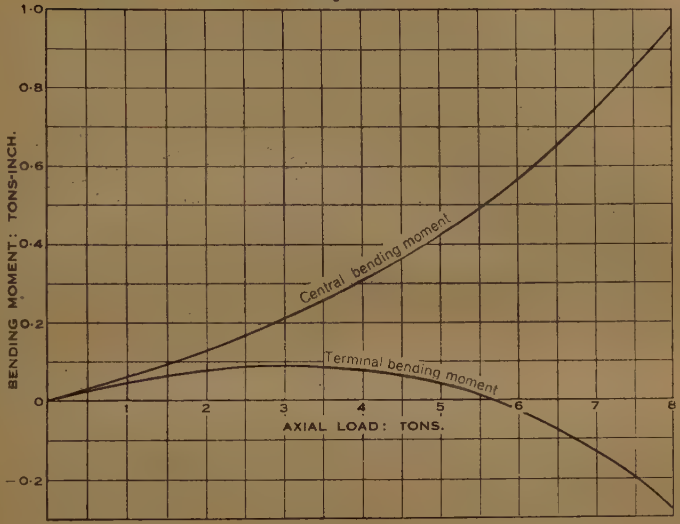
Failure will not occur until the extreme combined primary and secondary stress at the mid-cross-section of the member reaches the yield-point of the material, and the axial load which brings this about may be well beyond the first Euler limit. This point is illustrated by *Fig. 9* (p. 217), in which the extreme combined stress at the mid-cross-section is plotted in terms of the axial thrust.

For the particular case under consideration, if the yield-point of the material is 15 tons per square inch, this stress is not attained until the axial

TABLE II.

W	α	β	ϕ	Φ	Terminal bending moment: tons-inch.	Central bending moment: tons-inch.	Primary stress: tons per square inch.	Secondary stress at centre: tons per square inch.	Combined stress at centre: tons per square inch.	Central deflection: inch.
0	0	0	0	0	0	0	0	0	0	0
1	0·9351	0·5291	0·000121	0·000488	0·0252	0·0283	0·8197	0·1747	0·9944	0·0061
2	1·3223	0·7483	0·000241	0·000977	0·0464	0·0587	1·6393	0·3630	2·0023	0·0124
3	1·6196	0·9165	0·000362	0·001465	0·0631	0·0915	2·4590	0·5654	3·0244	0·0189
4	1·8701	1·0582	0·000482	0·001964	0·0757	0·1275	3·2787	0·7878	4·0665	0·0259
5	2·0909	1·1831	0·000603	0·002468	0·0835	0·1666	4·0984	1·0295	5·1279	0·0332
6	2·2904	1·2960	0·000724	0·002975	0·0863	0·2090	4·9180	1·2914	6·2094	0·0409
7	2·4740	1·3999	0·000844	0·003497	0·0838	0·2558	5·7377	1·5805	7·3182	0·0491
8	2·6447	1·4965	0·000965	0·004023	0·0754	0·3066	6·5574	1·8945	8·4519	0·0578
9	2·8052	1·5873	0·001085	0·004560	0·0606	0·3624	7·3770	2·2397	9·6167	0·0671
10	2·9569	1·6782	0·001206	0·005095	0·0426	0·4230	8·1967	2·6140	10·8107	0·0760
11	3·1012	1·7548	0·001326	0·005673	0·0099	0·4910	9·0164	3·1301	12·1465	0·0872
12	3·2391	1·8329	0·001447	0·006245	— 0·0276	0·5656	9·8360	3·4964	13·3324	0·0990
13	3·3714	1·9077	0·001568	0·006838	— 0·0745	0·6482	10·6557	4·0057	14·6614	0·1112
14	3·4987	1·9797	0·001688	0·007439	— 0·1312	0·7387	11·4754	4·5650	16·0404	0·1248
15	3·6214	2·0492	0·001809	0·008072	— 0·1998	0·8405	12·2951	5·1940	17·4891	0·1387
16	3·7402	2·1164	0·001929	0·008737	— 0·2820	0·9553	13·1147	5·9036	19·0183	0·1546

Fig. 7.

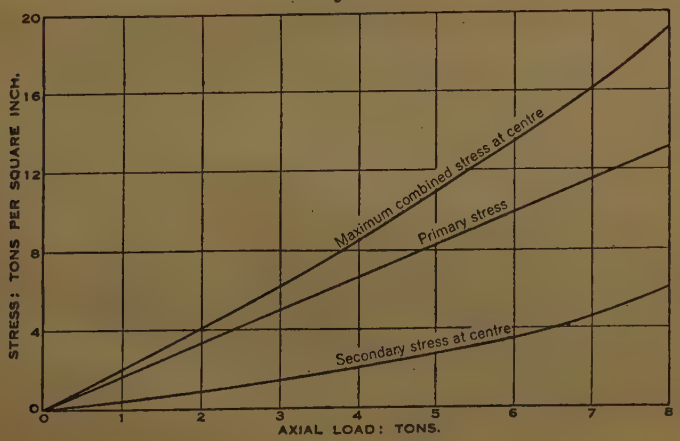


thrust reaches the value of 6·60 tons, which is well beyond the first Euler limit of 5·65 tons. Experimental verification of the analysis by which

Fig. 8.

the results recorded in *Figs. 7* and *9* were deduced is provided by *Fig. 10* (p. 218), in which the central deflection in the member is plotted in terms of its

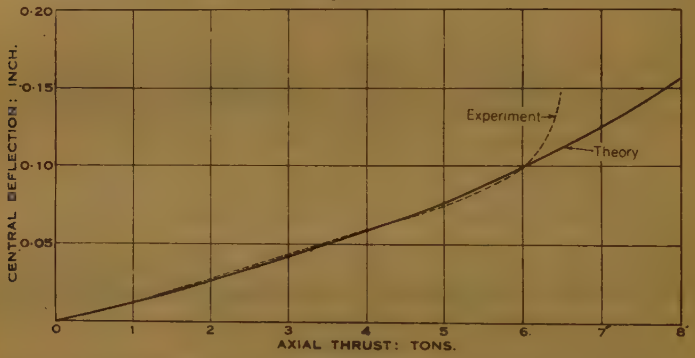
Fig. 9.



thrust. The full-line curve gives the deflection predicted by theory and the dotted curve gives the deflections actually obtained by experiment. The maximum discrepancy up to the point of failure is only of the order 0·0025 inch, and this is approaching the limits of instrumental accuracy.

The tensile yield-point of the material, which was annealed in a vacuum at 800° C., was found to be 13·5 tons per square inch, and it will be seen that indications of overstrain first appear when the axial thrust is about 6 tons. *Fig. 9* shows that the computed extreme stress at the mid-cross-section was then 13·3 tons per square inch, which again is a satisfactory confirmation of theory.

Fig. 10.



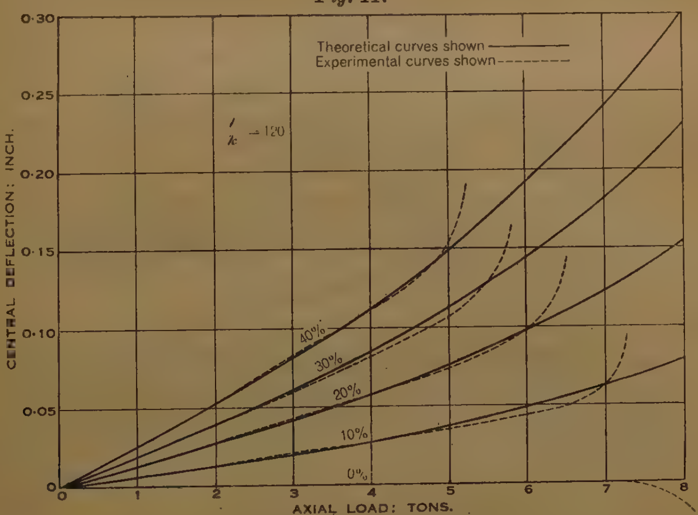
For the same framework, by altering the values of e, the effect of larger and smaller secondary bending moments were investigated. The results are recorded in *Fig. 11*, the percentages stated being the ratio of secondary to primary stress when the axial load is small. The full-line curve in each case gives the central deflection as predicted by theory, and the dotted curve gives the deflection which was obtained by experiment. The agreement in all cases is remarkably close, and, in accordance with expectation, the crippling load is diminished as the ratio of secondary to primary stress increases.

Similar theoretical predictions and experimental verifications were carried out with frameworks which made the slenderness-ratio of AB to be 60, 80, 100, and 140, and in all cases theory and experiment were found to be in close agreement. The results of these twenty-five investigations are set forth on *Fig. 12* (p. 220), which indicates how the crippling primary stress, determined experimentally, varied with the slenderness-ratio and the magnitude of the secondary stress, the percentages stated being the ratio of the secondary to primary stress when the loading is small. The crippling load recorded is in each case the greatest load which the strut could permanently sustain. In *Fig. 12*, the British Standards Institution formula for the safe primary stress in struts with riveted ends, namely $p = 9\left(1 - 0{\cdot}0038\,\frac{l}{K}\right)$ tons per square inch, and the corresponding formula

put forward by the American Railway Engineering Association, namely $p = 15{,}000 - \frac{1}{4}\frac{l^2}{K^2}$ lb. per square inch, are plotted.

These formulas for values of $\frac{l}{K}$ ranging from 60 to 140 are in close agreement, and it appears that even when an initial ratio of secondary to primary stress of 40 per cent. exists, the actual crippling primary stress is almost double that prescribed by these formulas. Consequently, if either of these formulas is adopted the margin of safety is such that there would appear to be no necessity to evaluate secondary stresses due to

Fig. 11.



stiffness of joints, unless it is expected that the ratio of secondary to primary stress as ordinarily calculated is likely to exceed 40 per cent.

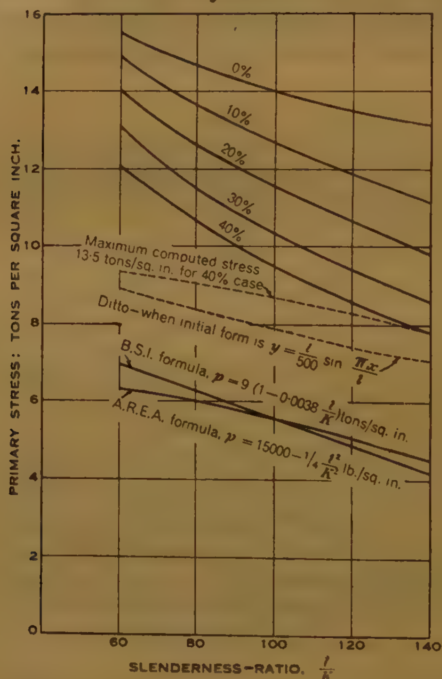
For the case where the initial ratio of secondary to primary stress is 40 per cent., the upper dotted curve in *Fig. 12* gives the computed primary stress in the strut which, for various slenderness-ratios, will stress the material to its yield-point of 13·5 tons per square inch. It will be seen that, whereas in the case of an exceptionally slender strut, $\frac{l}{K} = 140$, complete failure occurs as soon as the yield-point stress is reached, for shorter struts general failure does not occur until this limit has been considerably surpassed.

The lower dotted curve in *Fig. 12* shows how the computed primary stress which will induce in the strut a maximum stress of 13·5 tons per square inch is reduced if the strut, instead of being perfectly straight

initially, is bent slightly in the sinusoidal form $y = \frac{l}{500} \sin \frac{\pi x}{l}$, the initial ratio of secondary to primary stress again being 40 per cent. without taking the crookedness into account.

The foregoing calculations and experiments all deal with cases in which the ends of the strut under consideration are rotated in opposite directions.

Fig. 12.



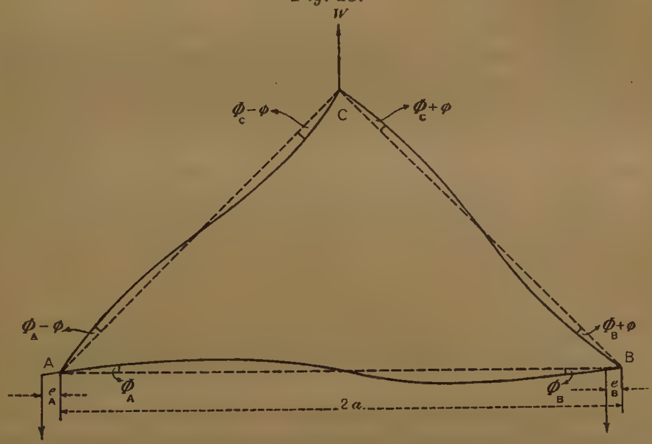
In actual structures this single-curvature bending of a compression member is somewhat exceptional, and more frequently bending is of the double-curvature variety, the ends of the strut being rotated in the same direction.

Double-Curvature Bending.

Since single-curvature bending, when it does occur, is likely to upset the stability of a strut much more than bending of the double-curvature character, the investigations recorded in this Paper deal mainly with the former; but a few experiments were performed to examine how the crippling load was influenced by secondary terminal couples which produced double-curvature bending.

The frameworks used in these supplementary tests were identical with those previously described and illustrated by *Fig. 6*, and by arranging the eccentricities e_A and e_B in the manner indicated in *Fig. 13*, the member AB could be tested under thrust combined with double-curvature bending of any desired degree of severity. If e_A and e_B denote the eccentricities of the loads at A and B, ϕ denotes the increase in the angles at A and B which is resisted by the stiffness of the joints, and Φ_A, Φ_B, and Φ_C denote

Fig. 13.



the counter-clockwise rotations of the joints A, B, and C, the general nature of the deformation of the framework will be as indicated by *Fig. 13.*

By considering the equilibrium of the joints, three equations can be laid down for finding Φ_A, Φ_B, and Φ_C, and, solving these, general expressions for Φ_A and Φ_B are obtained in the form :

$$\Phi_A = X + Y, \text{ and } \Phi_B = X - Y,$$

where

$$X = \frac{e_A + e_B}{2a\left[\dfrac{\tan\frac{\alpha}{2}}{\tan\frac{\alpha}{2} - \frac{\alpha}{2}} + \dfrac{2\tanh\beta}{\beta - \tanh\beta}\right]},$$

and

$$Y = \frac{e_A - e_B + 2a\dfrac{\tanh\frac{\beta}{2}}{\frac{\beta}{2} - \tanh\frac{\beta}{2}}\phi}{2a\left[\dfrac{2}{\alpha\tan\frac{\alpha}{2}} + \dfrac{1}{\beta}\times\dfrac{\beta\coth\beta - 1}{\frac{\beta}{2} - \tanh\frac{\beta}{2}}\right]}.$$

In these formulas $\alpha^2 = \frac{4a^2}{K^2} \times \frac{W}{2EA}$ and $\beta^2 = \frac{2a^2}{K'^2} \times \frac{W}{\sqrt{2}EA'}$.

The downward pulls at A and B resisting the upward force W at C are no longer $\frac{W}{2}$ precisely, but the discrepancy is quite insignificant, and has been ignored in establishing the general formulas for Φ_A and Φ_B.

Typical calculations relating to a case of double-curvature bending are set forth in Table III.

The slenderness-ratio of the member is 120, and the values taken for e_A and e_B, namely, 0·1455 and 0·3980 inch respectively, make the initial ratio of primary to secondary stress at each end 40 per cent., bending being of the double-curvature variety.

It will be seen from Table III that in this case there is no reversal in the direction of the terminal couples as the load increases, and it is found that for a given primary stress the maximum stress induced is considerably less than that developed in the corresponding case of single-curvature bending.

Thus for the five cases

$$M_B = -M_A,\ M_B = -\frac{1}{2}M_A,\ M_B = 0, M_B = \frac{1}{2}M_A,\ \text{and}\ M_B = M_A,$$

the initial ratio of secondary to primary stress at the end A being 40 per cent. in each case and the slenderness-ratio being 120, the values of the primary stress which induces an assumed yield-point stress of 13·5 tons per square inch are 8·3, 9, 9·8, 10·4, and 10·3 tons per square inch respectively.

Furthermore, with double-curvature bending yield will first be reached in sections situated at or near the ends of the member, and, on that account, final breakdown is not likely to be so sudden as failure in the case of single-curvature bending.

This point is brought out by *Fig. 14*, which records the experimental verification of the calculations set forth in Table III, the predicted and actual measurements being indicated by full- and dotted-line curves respectively.

Up to a primary stress of about 9 tons per square inch, theory and experiment are in very close agreement. For a primary stress of about 10 tons per square inch, yield at the end B is becoming noticeable, but complete failure does not take place until the primary stress reaches a value of about 11 tons per square inch.

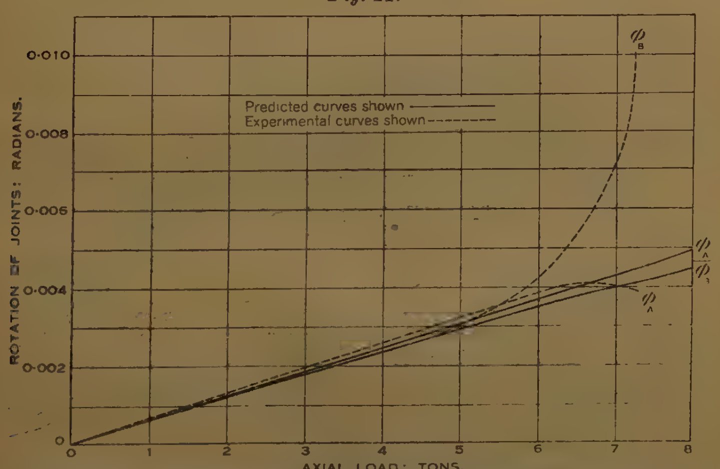
In comparison with the corresponding case of single-curvature bending recorded by the top dotted curve of *Fig. 11*, the loads producing initial yield and final breakdown are considerably raised and the last stages of breakdown are much more prolonged. For the three cases $M_B = -M_A$, $M_B = 0$, and $M_B = +M_A$, *Figs. 15*, *16*, and *17* (pp. 224–225) give the

TABLE III.

W	Φ_A	Φ_B	Terminal bending moment: tons-inch. M_A	Terminal bending moment: tons-inch. M_B	Primary stress: tons per square inch.	Maximum secondary stress: tons per square inch.	Maximum total stress: tons per square inch.
1	0·000315	0·000313	0·0522	0·0521	0·8197	0·3227	1·1424
2	0·000629	0·000635	0·1020	0·1018	1·6394	0·6306	2·2700
3	0·000942	0·000928	0·1500	0·1495	2·4591	0·9273	3·3864
4	0·001251	0·001228	0·1961	0·1952	3·2788	1·2123	4·4911
5	0·001562	0·001525	0·2399	0·2387	4·0985	1·4831	5·5816
6	0·001871	0·001817	0·2818	0·2803	4·9182	1·7421	6·6603
7	0·002162	0·002107	0·3216	0·3199	5·7379	1·9881	7·7260
8	0·002441	0·002392	0·3672	0·3679	6·5576	2·2744	8·8320
9	0·002795	0·002675	0·4041	0·4045	7·3773	2·5006	9·8779
10	0·003105	0·002965	0·4301	0·4290	8·1970	2·6589	10·8559
11	0·003413	0·003232	0·4608	0·4605	9·0167	2·8487	11·8654
12	0·003722	0·003506	0·4883	0·4892	9·8364	3·0279	12·8643
13	0·004033	0·003777	0·5178	0·5205	10·6561	3·2387	13·8948
14	0·004347	0·004042	0·5426	0·5480	11·4758	3·4428	14·9186
15	0·004657	0·004311	0·5655	0·5741	12·2955	3·6536	15·9491
16	0·004973	0·004574	0·5852	0·5981	13·1152	3·8693	16·9845

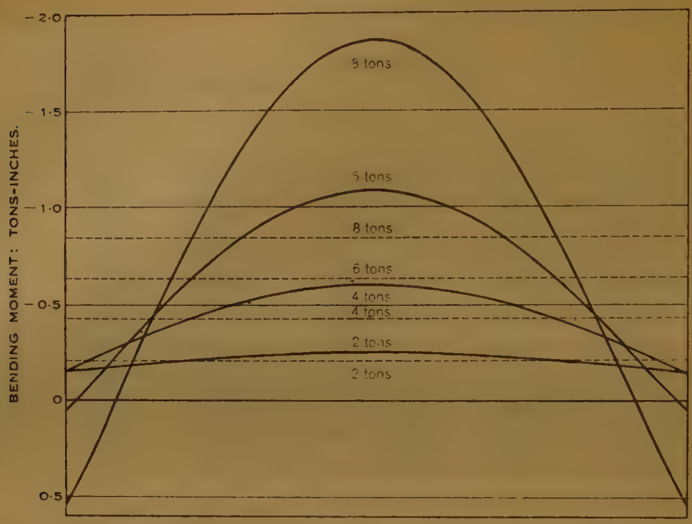
bending-moment diagrams in the member AB for various values of the axial load, $\frac{l}{K}$ being 120 and the initial ratio of secondary to primary stress at the end A being 40 per cent. in each case. The dotted lines give the corresponding results which are obtained if change in flexibility and deformations due to axial load are ignored.

Fig. 14.



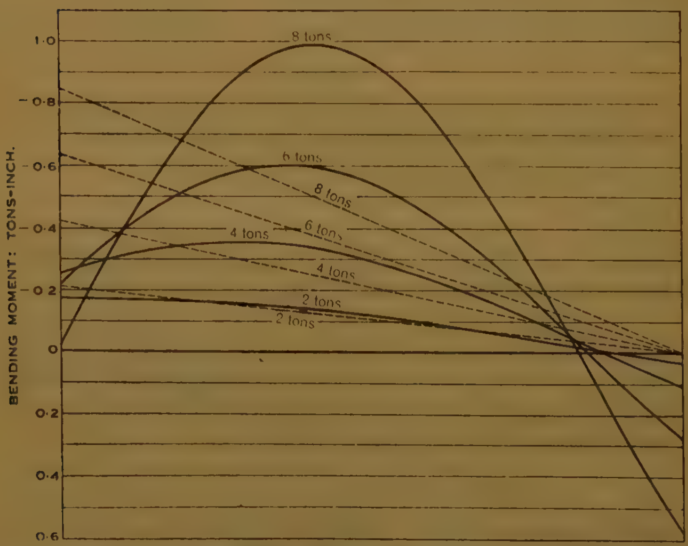
EXPERIMENTAL VERIFICATION OF CALCULATIONS SHOWN IN TABLE III.
($l/k = 120$. For light loads the ratio of secondary stress to primary stress is 40 per cent., and $M_B = M_A$.)

Fig. 15.



BENDING-MOMENT DIAGRAMS FOR COMPRESSION MEMBER AB.
(l/k = 120. For light loads the ratio of secondary stress to primary stress at the end A is 40 per cent., and $M_B = M_A$.)

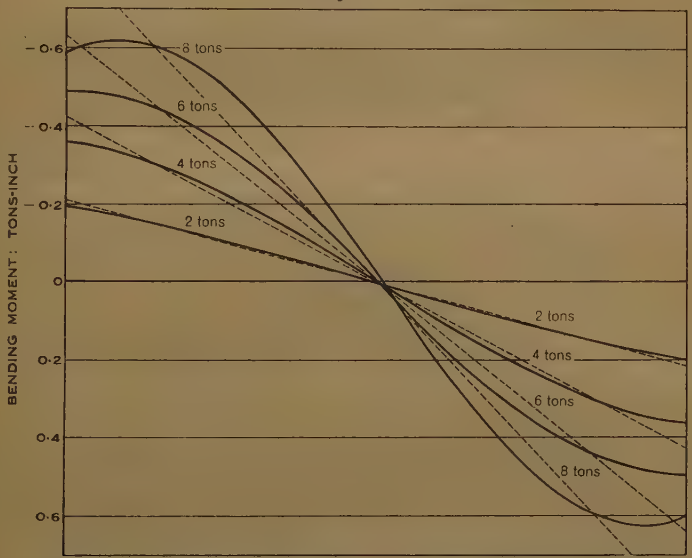
Fig. 16.



BENDING-MOMENT DIAGRAMS FOR COMPRESSION-MEMBER AB.
(l/k = 120. For light loads the ratio of secondary stress to primary stress at the end A is 40 per cent., and $M_B = O$.)

In *Fig. 15*, which represents the case of single-curvature bending ($M_B = -M_A$), except for the smallest axial loads, the results given by the dotted lines have little relation to the truth, and the terminal bending moments they predict are not merely erroneous in magnitude but are even wrong in direction. In *Fig. 16*, which represents the case when $M_B = 0$, the results given by the dotted lines are still far removed from the truth, but in *Fig. 17*, which illustrates the example of double-curvature bending

Fig. 17.



BENDING-MOMENT DIAGRAMS FOR COMPRESSION-MEMBER AB.
($l/k = 120$. For light loads the ratio of secondary stress to primary stress is 40 per cent., and $M_B = +M_A$.)

($M_B = +M_A$), a satisfactory agreement is revealed, and maximum bending moments, as predicted by the "constant-flexibility" method, although not strictly accurate, err at any rate on the safe side.

CONCLUSION.

In triangulated frameworks, more often than not, four members radiate from a joint, whereas in the frameworks studied in this research a joint is held by two members only, the action on it of other members being represented by a couple whose magnitude was directly proportionate to the applied load. Owing to variations in their flexibility, the couple introduced by additional members will not, in general, vary in this manner, but the departure from reality will not be great, particularly where one of the additional members is a strut and the other a tie, since in such a

case the increased flexibility of the strut will, to some extent, be balanced by the decreased flexibility of the tie. It is proposed to apply theory and experiment to a framework of a more elaborate character in which four members radiate from some of the joints, but it is more than likely that this further investigation will merely confirm the main conclusions relating to the terminal bending moments and crippling loads reached in the case of an isolated triangular framework. These main conclusions are as follows :—

(1) The terminal bending moments in the members of a framework with stiff joints may be greatly influenced by changes in flexibility due to axial loading; this is particularly the case when the members are slender and are loaded nearly up to the point of failure.

(2) Owing to the change in flexibility due to axial loads, the principle of superposition does not apply. Terminal bending moments are not proportionate to the applied loads, and as the loading of a framework increases they may actually diminish and change sign.

(3) Secondary terminal bending moments which act in the same direction are less destructive to a compression-member than these same terminal bending moments acting in opposite directions.

(4) The B.S.I. formula, $p = 9\left(1 - 0{\cdot}0038\frac{l}{K}\right)$ tons per square inch, or the A.E.R.A. formula, $p = 15{,}000 - \frac{1}{4}\times\frac{l^2}{K^2}$ lb. per square inch, prescribing the safe axial load for a compression member with riveted ends, provide a margin of safety sufficiently wide to accommodate the secondary stresses which are likely to be induced in any ordinary stiff-jointed framework.

Acknowledgements.

The Author records with gratitude the valuable assistance received from Mr. E. H. Lee, B.A., Scholar of Caius College, Cambridge, and Mr. A. L. Percival, M.A., Fellow of Jesus College, Cambridge, in carrying out computations and experimental verifications.

Paper No. 5223.

"The Hydrology of the Yangtze River."

By HERBERT CHATLEY, D.Sc. (Eng.), M. Inst. C.E.

(Ordered by the Council to be published with written discussion.) [1]

TABLE OF CONTENTS.

GENERAL.

ALTHOUGH the regulation of the Yangtze River against floods has been carried on empirically for about 1,500 years, it was not until 1915 that any measurements were made which would serve as an exact basis for design. In that year the Whangpoo Conservancy Board, under the initiative of the then Swedish Engineer-in-Chief, Lieutenant-Colonel H. von Heidenstam, M. Inst. C.E., arranged with the Chinese Maritime Customs for a special survey and gaugings in the estuary, and also measured the discharges at a point, Wuhu, some 200 miles up from the mouth, where the tidal action is slight. Previously, sextant traverse surveys of the navigable channels, extending over about 1,400 miles, had been made by the British Admiralty, and had been revised by the Chinese Maritime Customs, but no complete cross-sections, levellings, or gaugings had been made. Daily water-levels, referred to local zeros, had been recorded by the Customs at about ten places along the river for about 50 years, and some rainfall data had also been collected by the Customs and by the Jesuit missionaries. In 1880, Dr. H. B. Guppy had made approximate valuations of the discharge and the silt content. The 1916 survey was succeeded in 1919 to 1921 by a special study of the estuary and lower river (including a geological survey) up to Wuhu. The results of this work done between 1915 and 1921 are given in the Reports published by the Whangpoo Conservancy Board. In 1922, the Yangtze River Commission was formed with the principal aim of improving navigational depths up to Hankow (600 miles from the sea), but in 1923 the late Sir Frederick Palmer, Past-President Inst. C.E., reported that such improvement would not be economically warranted,

[1] Correspondence on this Paper can be accepted until the 1st August, 1940, and will be published in the Institution Journal for October 1940.—SEC. INST. C.E.

and for the next few years the Commission occupied itself (in conjunction with the newly formed Hydrographical Bureau of the Chinese Admiralty) with rain measurements, levelling, sounding, gauging, and mapping. In 1931 there was an unusually severe flood and the Commission co-operated vigorously with a special Flood Relief Commission in the repairing and raising of the dykes. Some 90,000 square kilometres had been seriously flooded, and many millions of cubic metres of earthwork were rebuilt. For political reasons attention then reverted to the question of navigable improvements. Grandiose schemes of training were developed on paper, and, in the preparation of these, further useful data as to depths, currents, etc., were gained. The Commission assisted territorial authorities in various specific reclamations and training works of a local character. The Yangtze Commission published a series of reports in Chinese and English from 1922 to 1932, but, unfortunately, after 1932, the work is either unpublished or reported only in Chinese. The name of G. G. Stroebe, an American engineer, should be specially noted in connexion with the survey works. Colonel Heidenstam was a member of the Technical Committee of the Commission for 1922 to 1927, as was the Author from 1928–1932. The Author was also a member of the Technical Committee of the Flood Relief Commission in 1931, and, as a consultant in the years 1932 to 1937, reported on the Yangtze on several occasions to the Chinese Government. There was always co-operation between the Whangpoo Conservancy Board (to which the Author was Engineer-in-Chief from 1928–37), the Chinese Maritime Customs, the Hydrographical Bureau, and the Yangtze River Commission.

While many matters concerning the river are still obscure, so much information has now been gained that a summary of the principal results will probably be useful to all students of hydraulics.

Topography of River.

Fig. 1, Plate 1 shows a general plan of the alluvial plain of the river, within which most of the survey work has been done. It is interesting to note how the narrow alluvial plain is shut in by high ground and how the river is constricted at numerous places by rocky foreshores and hills.

Figs. 2, 3, and 4, Plate 2 show the principal slopes, widths, and sectional areas, plotted on the " thalweg " line.

Below the riverport of Chinkiang, about 250 kilometres from the mouth, the river is normally augmented by the discharge of the Hwai river, a relatively small but heavily silt-laden stream lying between the Yangtze and the Yellow river. This peculiarity is due to the dykes of the transversely placed Grand Canal and the past migrations of the Yellow River, which have blocked the normal discharge of the Hwai to the sea. At the time when hostilities broke out in 1937, certain large regulative works for

the Hwai River and Grand Canal were well advanced, and a small relief channel to the sea is open.

Two lakes, the Tung Ting, south-west of Hankow, and the Po Yang, south of Kiukiang, act to some extent as storage reservoirs for the Yangtze or its tributaries, but even so, there is a very great difference in the summer and winter levels, and throughout the alluvial plain the summer water is kept off the plain by rather high dykes. The dykes are not well placed, are of irregular trace, and are very little protected against scour, but their collective volume is enormous, although less than that of the Yellow River dykes. The numerous hills prevent any large wandering of the stream, and its instabilities are local rather than general. Nevertheless, there is a good deal of change and, at some places, bank erosion proceeds at a very high rate.

The delta and estuary are moderately stable, but the low-water line advances seawards at a rate which is of the order of 1 mile in about 40 or 50 years. Very powerful tidal currents operate in the estuary, running as strongly as 6 knots at spring tides on the entrance bars.

Hydrology.

The principal source of silt is the so-called "Red Basin" in Szechuan, but all the tributaries bring down a good deal of sediment and the estuarial shoals grow at a rate of about $1\frac{1}{2}$ square miles per year. The sea-bed falls quite slowly towards Japan and a long spur of submarine deposits can be traced about two-thirds of the way across. The water in the river is always coffee coloured as far as an oscillating tidal margin seaward of the estuary, but the silt content very rarely rises above 0·2 per cent. by weight, far less than that of the Yellow River. The silt content of the Yangtze in 1930 is shown in Table I, p. 230. It does not appear that there is very much bottom drift, but, owing to the opacity of the water and the huge dimensions of the estuary and sea approaches, there is some doubt on this point.

The Yangtze "gorges", which terminate above the alluvial plain, have been studied to some extent. The mean water gradient is about 1 foot per mile. Some of the rapids are through syenite "dykes." The water stages at several points have been observed for many years (a range of about 200 feet between extraordinary high and extraordinary low water has been recorded at one place), levels have been taken and the topography has been surveyed (but not in great detail), and rather general soundings have been taken (see Table II, p. 230). The depths vary from 200 feet in the pools to very little at the rapids, and it is known that currents of ten knots or more occur at certain rapids, but no gaugings have been made. The gaugings at Ichang just below the gorges are, of course, indicative of the discharge in them.

The area of the whole watershed is reckoned to be about 2,000,000 square kilometres, but as no exact survey has been made of the western-

TABLE I.—SILT IN SUSPENSION IN YANGTZE, 1930.

Station.	Maximum recorded: parts per million.	Date.	Minimum recorded: parts per million.	Date.	Remarks.
Shasi (1,605 kilometres above Woosung).	453	21 May	49	5 February	Short record, not including summer months.
Various places from Hankow down to Wuhu.	1,553	25 August	85	23 January.	Mean values, top to bottom.
Wuhu	1,200	September	20	February	At 20 feet below surface, for four years.
Kiangyin, in upper part of estuary, 150 kilometres above Woosung.	300	July	50	March	Monthly averages at 20 feet below surface, for 4 years. Semi-tidal.
Mouth of Whangpoo at Woosung.	480	January	150	November	Ditto.
Ditto.	1,200	Winter Spring-tides.	7	March neap-tides.	At 20 feet below surface, slack after high-water.
Entrance bar (50 kilometres below Woosung.	2,000	Winter Spring-tide.	200	Neap-tide.	Ditto. Very strong tidal currents up to 6 knots at Spring-tides.

most areas (even the northern and southern divides have only been partially surveyed), there is a margin of error in this of perhaps 5 per cent. This total includes the watershed of the Hwai River (170,000 square kilometres). It is divided up as shown in Table III.

TABLE II.—LEVELS IN THE YANGTZE VALLEY.

Place.	Elevation of gauge zero: metres.	Distance above Woosung: kilometres.	Maximum rise, 1931: metres.	Remarks.
Woosung	0·00	0	5·79	50 kilometers above entrance bar. Tidal.
Chinkiang	1·51	250	6·40	Semi-tidal.
Nanking	1·67	375	7·62	Partially tidal.
Wuhu	2·33	467	9·45	Slightly tidal.
Kiukiang	6·66	817	13·72	
Hankow	11·94	1,130	16·46	
Shasi	32·88	1,606	10·67	
Ichang	39·69	1,735	17·68	Head of plain.
Wanhsien	99·09	2,055	39·01	Gorges.
Chungking	166·54	2,383	26·52	
Suifu	(330·00)	(2,700)	(30·48)	Approximate.
Batang	(2,500·00)	(4,000)	—	Approximate.
Source in Tibet	(5,000·00)	(4,800)	—	Approximate.

Precipitation has been measured in the lower main valley with considerable accuracy, but in the mountain areas (especially in the remote west) the data are scanty or are for short periods only. In the alluvial plain the annual average is over 30 inches, but towards the Tibetan border this diminishes to about 15 inches. The snow measurements in the Western mountains are insufficient, but the general evidence indicates that snow is a minor factor, the bulk of the precipitation being monsoon rain in the central and eastern parts of the watershed. The fluctuations of rainfall from the averages are very considerable, say, 50 per cent. annually and 100 per cent. monthly. Maximum rainfall is of the order of 12 inches

TABLE III.—WATERSHED OF THE YANGTZE.

District.	Approximate area: square kilometres.	Approximate average rainfall: millimetres.	
		Annual.	Worst month.
Above Suifu [limit of navigation].	480,000	750	150
Suifu to lower end of gorges .	540,000	1,100	200
Hsiang River and T'ung T'ing Lake.	250,000	1,300	230
Han River	180,000	1,100	280
Kan River and Po Yang Lake.	180,000	1,600	310
Hwai River	170,000	1,000	180
Smaller tributaries in lower river.	150,000	1,100	200
Total . . .	1,950,000		

in 24 hours. The Hankow area is particularly liable to flooding owing to the entrance of two large tributaries, one at Hankow and the other closely above that place.

The ratio, $\frac{\text{run-off}}{\text{precipitation}}$, for the whole river is of the order of 30 per cent. It is probably less in the plain for ordinary precipitation and more in the mountains, but the evaporation losses from the lakes and river surfaces equalize matters to some extent.

Evaporation measurements have been taken in pans exposed on water at various places. In the main valley the annual totals were 1,000 to 1,200 millimetres with a monthly maximum (July or August, mean daily maximum temperature about 85°) of about 200 millimetres. The absence of convection currents in pan readings makes their value somewhat uncertain, but it is obvious that areas of water can lose by evaporation nearly as much as the rainfall which comes directly upon them. The total evaporation from an intermittently wet surface is, of course, less than the pan value. The growing of rice in the alluvial plain promotes rapid transpiration, so

that, in the rainy season, rice areas also vaporize almost all the rain which falls on them, except during unusually heavy rainfall. Even the latter is partly dealt with by storage in myriads of ponds and canals.

In the great flood of 1931 the discharge at Hankow is considered to have been over 70,000 cubic metres per second, and if the dykes had not failed the water-line there would have been about 1 metre higher than it actually was. Maximum and minimum discharges at other points are shown in Table IV.

TABLE IV.—MAIN STREAM: DISCHARGES AND MEAN VELOCITIES.

Station.	Maximum recorded: cubic metres per second.	Mean velocity: metres per second.	Date.	Minimum recorded: cubic metres per second.	Mean velocity: metres per second.	Date.
I-chang	28,780	1·88	14/10/31	—	—	—
Szepakow (arm near Shasi).	21,000	—	9/7/25	4,820	—	18/2/26
Chenglingki (above Hankow).	54,850	1·35	5/8/31	6,530	—	3/2/26
Hankow . . .	60,750	1·80	29/7/24	5,208	—	26/1/23
Kiukiang . . .	64,350	1·83	8/9/31	4,818	—	9/2/25
Hukow (near Kiukiang).	65,880 59,890	— 1·45	8/7/24 10/9/31	5,595	—	30/1/23
Tatung (between Wuhu and Kiukiang).	67,670	1·56	1/9/31	7,721	—	17/1/23
Wuhu	61,500	1·95	27/7/15	7,260	0·37	24/2/16
Kiangyin . . .	Ebb 64,494	1·447	6/9/32	Flood (up) 24,000	0·57	28/3/15
Tsungming Island, upper end above Woosung.	Ebb 130,000	1·00 (approximately)	23/9/15	Flood (up) 120,000	0·80 (approximately)	24/9/15

Prevalent winds are from the south-east (north-east in winter). Severe continental storms occur on the coast in the winter and typhoons in the late summer. The latter may penetrate to the central areas.

The silt originates above the alluvial plain, but there is a great deal of sediment eroded at bends and redeposited in the shallows. It is very fine (0·1 millimetre or less in diameter) especially in the estuary, but a certain quantity of coarser sand (up to about 1 millimetre) is sorted out at certain places.

At the crossings there are important annual changes. Generally speaking the crossings deepen as the river rises and re-accrete as the river falls, the navigable depths being thus doubly affected. There are about a dozen places between Hankow and the mouth where the greatest depth of water is only about 10–12 feet at low stage, and between the head of the plain and Hankow the maximum depths in the minimum crossings may be as little as 6 feet. This latter part is also very tortuous and unstable. There

are four main tributaries entering the plain: the Hsiang at Yochow; the Han at Hankow; the Kan at Kiukiang; and the Hwai at Chinkiang. The Whangpoo enters at Woosung 30 miles above the main entrance bar and is of no real importance in relation to the river as a whole, but does provide protected berthage for shipping at the nearest point to the sea, whence comes the great importance of Shanghai as a trading centre.

The Paper is accompanied by two sheets of diagrams, from which Plates 1 and 2 have been prepared, and by the following Appendix.

[APPENDIX.

APPENDIX I.

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THE HYDROLOGY OF THE YANGTZE RIVER.

Fig: 1.

THE YANGTZE ALLUVIAL PLAIN.

William Clowes & Sons, Limited: London.

H. CHATLEY.

Paper No. 5221.

"Derailments of Four-Wheeled Stock on the Northern Section of the Assam–Bengal Railway. Experiments in the Running and Lateral Oscillation of this Four-Wheeled Stock."

By Frank James Salberg, M.B.E., V.D., M. Inst. C.E.

(Ordered by the Council to be published in abstract form).[1]

After a series of nine unaccountable derailments of four-wheeled wagons, a large number of test runs were made with a Hallade recorder on different classes of track, with different types of four-wheeled wagons variously loaded, and at speeds ranging from 20 to 45 miles per hour. An analysis of the one hundred and seventy odd Hallade records showed, beside much negative information :—

(1) That lateral oscillation was always present and always periodic, irrespective of the track and of the magnitude of oscillation. The following empirical formula covers the range of observation :—

$$P = \frac{89 - V}{55},$$

where P denotes the period of oscillation in seconds, and V the speed in miles per hour.

(2) That the magnitude of oscillation was affected by track conditions, but was irrespective of rolling.

The Author ascribes all these unaccountable derailments to a large oscillation exactly synchronizing with a wheel that had but little or no weight on the rail. It is explained, following Stockhammer[2], how, with these wagons, the existing wagon-department practice of permitting a variation of $\frac{1}{4}$ inch in the spring-camber is equal to a difference of $1\frac{1}{4}$ inch out of cross-level in a 30-foot length of track (metre gauge). It would appear as though greater attention is required in the more exact standardization of wagon springs, and that very particular care is necessary in the assembly of a wagon on its axles.

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[1] The MS. and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

[2] G. Stockhammer, "The Derailment of Railway Carriages." From Bulletin of the International Railway Congress, May 1899. Reprinted by Government of India. Technical Paper No. 80. Delhi, May 1900.

It is generally accepted that the coning of railway tires helps to keep the axle centred on the track and the flanges away from the rails, but it is perhaps not so generally recognized that this centring of the wheels is not about the centre of the track, but is about the centre of the points of contact of the treads with the rails. After the passage of a train over rails that have been rust-dimmed, it is noticeable that the path of the tread-contact winds down the rail-table, sometimes nearer one side and sometimes nearer the other. It is from this feature that the Author formulated the following theses :—

(1) That a coned-tire pair of wheels and axle must be perpetually hunting in attempting to centre itself exactly on the virtual gauge, the virtual gauge being the points of contact of the wheel-treads on the rail-table. Considerable variation can be caused in the virtual gauge by uneven canting of the rail.
(2) That the wagon-body with its load floating on the journals acts like a bifilar pendulum, and its period of oscillation is dependant on the moment of inertia of its plan.
(3) That every movement on the rail of the coned-tire axle centring itself necessarily causes a reaction at the brasses, for the axle in centring itself would get askew to the track unless resisted.
(4) The theory, therefore, is that the frequent reactions at the brasses of the axles centring themselves on the virtual gauge keep the wagon-body in a bifilar type of pendulum movement; and that the impulses are of insufficient magnitude to affect the period of oscillation, which is dependent on the moment of inertia of the wagon-body, but are sufficient to alter the amplitude of oscillation, and may, if they occur at correct phase, create very large oscillations.

To confirm the practical issues of this theory further tests were carried out and showed that the tire-movement on the rail-table was entirely irregular, and was of the order of ¼ inch.

The final series of tests consisted of running one wagon with the following combinations :—

(*a*) Standard 1-in-20 coned tires with standard journals.
(*b*) Cylindrical tires with standard journals.
(*c*) 1-in-100 coned tires with standard journals.
(*d*) Standard 1-in-20 coned tires with corrugated journals.
(*e*) Cylindrical tires with corrugated journals.
(*f*) 1-in-100 coned tires with corrugated journals.

It is in the results of this series of tests that the Author finds the substantiation for the theory advanced, for in (*a*), (*b*), and (*c*) the oscillation of the wagon-body was still periodic and the periods agreed with former experiments : in (*a*) the amplitude of the oscillation was nearly, as before,

0·5 inch, in (*b*) and (*c*) the amplitude was halved to 0·22 inch. In (*d*), (*e*), and (*f*) (with corrugated journals) the records of all three tires showed an amplitude of oscillation of from 0·1 to 0·11 inch, which amplitude, as would be expected, was roughly half the normal ¼-inch movement of the tire on the rail. This also shows that the wagon had not used any of its axle-guide play, an important feature in demonstrating that a bifilar type of movement was not present. Remarkable results were obtained with the 1-in-20 coned tires and corrugated journals, and show that this method holds out the prospect of complete elimination of the periodic oscillation from which all four-wheeled stock suffers. Vehicles with this type of journal are in use on two other railways in India.

The Author considers that the results of these investigations have a very much wider application, and he believes the theory that the oscillation caused by the coned tires centring their axles on the virtual gauge is most probably the reason for trouble with bogie stock at high speeds, and possibly may be the cause of dangerous "hunting" with certain classes of engines.

All tests, records, and work in analysing the results were carried out by Mr. F. E. Musgrave, M.C. Mr. Musgrave was also responsible for the devices rigged in the final test wagon, and he was assisted throughout by draughtsman H. S. Guha, whilst he had the active co-operation of many other Assam–Bengal Railway officers in the course of these tests.

"Coventry By-Pass Road."

By Ernest Hone Ford, M. Inst. C.E., M. Inst. M. & Cy. E.

(*Abridged.*)[1]

The Coventry by-pass road, although it is 150 feet wide, cannot be regarded as a "motorway." It is, rather, a semi-developed road, adapted for traffic according to modern conceptions. Traffic of all classes on the Birmingham–London route through Coventry had, in 1935, increased by 520 per cent. of its total in 1922.

The Parliamentary powers obtained by the Corporation in the Act of 1930 involved a new and interesting proposal of a charge on the frontage tenants of a sum of 30*s*. per foot of frontage, as and when building occurred. Betterment was to be set off against the compensation payable for the land required. In 1935, further Parliamentary powers had to be obtained, owing to the acceptance by the City Council, and the Ministry of Transport, of the proposal of dual carriageways, cycle-tracks, and service roads. The Ministry of Transport decided to grant 75 per cent. towards the cost of the road, without the previous conditions referring to the employment of labour from distressed areas.

The approximate costs of the work, including the construction of service roads, bridges, and all incidental works, are shown in Table I.

Table I.

Item.	Cost: £.	Assistance.
Main road	259,000	Grant: 75 per cent.
Land acquisition	30,000	Grant: 75 per cent.
Service roads	42,000	Recoupment of 30*s*. per foot of frontage.
Tile Hill Lane section (estimated)	19,000	Grant: 75 per cent.
Total	350,000	

An amount of £30,000 must be added for existing 80-foot widths, partially reconstructed.

The length of the by-pass road was 6⅞ miles, whilst the length of the old route through the city was 6½ miles. There were seven islands, 90–110 feet in diameter, whilst the island at the London Road junction was 180 feet in diameter.

[1] This Paper was read before a Joint Meeting of the Birmingham and District Association of The Institution and of the West Midland District Branch of the Institution of Municipal and County Engineers on the 26th January 1940. It was published in Journal Inst. M. & Cy. E., vol. 66, p. 596 (27th February 1940).

The concrete type of carriageway was adopted wherever practicable since the cost of the stone-pitched road surfaced with asphalt-macadam was 30 per cent. greater. The concrete was laid in slabs 12 feet wide and 48 feet 6 inches long in alternate bays, 7 days elapsing before the laying of the intermediate bays.

Where a stone-pitched carriageway was adopted, the usual specification of 9 inches of pitching and 4 inches of ashes was applied. The water-bound macadam laid on the pitching, to provide a surface for 12 weeks pending the laying of asphalt-macadam, was later abandoned in favour of bitumenized tar-macadam, as it was found that the water-bound macadam, though sealed with cold emulsion, did not stand up to traffic for any length of time and a considerable amount of repair work was constantly involved.

Wherever possible, the excavation was carried out by means of mechanical tractor-drawn scrapers of the Le Tourneau type, mainly of 8 cubic yards capacity. These machines gave a very high degree of consolidation to the filling. Considerable use was made, also, of bull-dozer plant in regulating and grading the filling, particularly in carriageway formations.

For consolidating the asphalt-macadam surfacing, a 14-ton roller of the most modern type was used. This had a small hydraulically-operated plane roller (between the two main drums) which was very effective for finishing, and was only used in the final rolling. The macadam was finished with a layer of pre-coated chippings.

Cycle-tracks were constructed in tar and limestone, except at road junctions, where an apron of concrete was laid in order to show up the entrance to the cycle-track. The tracks are bordered by a concrete curb, 10 inches by 2 inches.

The total amount of material excavated and the total amount of filling were each 400,000 cubic yards, whilst 35,000 tons of stone was used for pitching. The maximum height of embankment is 17 feet, and the maximum depth of cutting was 14 feet, whilst the steepest gradient was 1 in 20. A length of 4 miles of the carriageway was pitched, $2\frac{7}{8}$ miles concreted, $\frac{3}{4}$ mile coated with tar-macadam (temporary surface), and $4\frac{3}{4}$ miles coated with asphalt-macadam. The maximum number of men employed at any one time was 250, and the work required $3\frac{1}{2}$ years to complete.

The electrical engineer's estimate for lighting was £4,200. The standards are 25 feet high, are spaced 200 feet apart, and carry 250-watt mercury-vapour gaseous-discharge lamps.

Tree and hedge planting was carefully designed to suit: (1) the traffic; (2) the various soils met with; and (3) the type of district through which each section passes. 1,608 trees of thirty-eight varieties, and $2\frac{1}{2}$ miles of privet were planted.

Two bridges, one over the river Sowe, the other over the river Sherbourne, are each 110 feet wide and have 24-foot dual carriageways, 8-foot cycle-tracks, and 18-foot grass centre strips.

The bridge over the river Sowe is founded on 13-inch-diameter "Vibro" concrete piles and has two 24-foot end spans and a 28-foot 6-inch middle span. Piles were to be driven to a minimum penetration of 16 feet, or until the penetration caused by the equivalent of five successive blows with a 2-ton drop-hammer falling 3 feet did not exceed 1 inch, whichever was the greater. In practice, the penetration varied from 21 feet to 30 feet 6 inches. The "Vibro" piling hammer is fitted with extracting links. The minimum frequency of blows was specified at 20 per minute. The piles, which had main reinforcement of four $\frac{3}{4}$-inch-diameter bars, were driven, and the excess length was cut off, before the excavation for the abutment walls was commenced.

A test of a pile for safe loading was made by means of a hydraulic jack. The pile was tested successfully with a 50 per cent. overload. The ultimate bearing capacity from the driving was 103 tons, giving a factor of safety of 4 over the working load of 25 tons.

The decking is a continuous reinforced-concrete slab, 1 foot $7\frac{1}{2}$ inches thick. The piers are of the open type with columns, 18 inches by 15 inches, spaced 7 feet 10 inches apart to which the loads are transmitted by top sill beams. The parapets are in white concrete with panels recessed 2 inches. At each pilaster there is an expansion-joint above the piers, and the coping consists of double bull-nose pre-cast white concrete blocks.

The bridge over the river Sherbourne is a single skew span of 47 feet 6 inches, crossing at an angle of 42 degrees 30 minutes. It has a beam-and-slab deck and a 8-inch slab and ribs at 6-foot 9-inch centres under the road. The beams are 1 foot 6 inches wide by 3 feet 10 inches deep. The abutments are of the cantilever type, and training walls are built along the river banks. The parapet is similar to that on the river Sowe bridge.

Rocker bearings for the pre-cast reinforced-concrete beams in the Sherbourne bridge consist of mild-steel plates, 14 inches by 9 inches, and a cast-steel rocker 12 inches by 6 inches. The rockers are set in steel cork-lined boxes. While the beams were being placed the rockers were held in position by temporary wedges.

The Green Lane railway bridge was on the route of the by-pass road, and it had to be entirely reconstructed and widened from 26 feet to 85 feet (the span being 30 feet). The abutments were extended in stone corresponding to the original stone, and the decking was formed with multiple compound steel girders at 4-foot 10-inch centres and brick jack-arches. A temporary bridge was provided, clear of the job, to carry traffic during the progress of the works.

The Fletchamstead railway bridge is a new bridge constructed to carry the by-pass road over the London–Birmingham line. It is 60 feet in span and 80 feet wide. The abutments, wing-walls, and parapets are of brick-work corresponding in colour to the factories in the neighbourhood. The decking is made up of compound steel girders spaced at 6-foot 3-inch centres. Brick arches have been constructed between these girders.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Committee on Earth Pressures: Experiments Panel.

Report on Soil-Mechanics Research, received from the Building Research Station.

In the field of soil-mechanics one of the outstanding events of the past year was the Forty-Fifth James Forrest Lecture by Dr. Karl von Terzaghi, M. Inst. C.E., who chose for his subject "Soil Mechanics—A New Chapter in Engineering Science."[1] After giving a masterly review of the present position of this new branch of engineering science, Dr. von Terzaghi concluded his remarks by emphasizing the necessity for close co-operation between the soils-laboratory and the practising engineer if the benefits of the information which can be derived from soil-research are to be fully utilized, and if the new science is to take its proper place in civil engineering in Great Britain.

This aspect of soil-mechanics research has been fully appreciated by the Experiments Panel of the Earth Pressures Committee, and during the past year part of the work of the Panel has been directed towards establishing this co-operation. As part of the scheme of work which is being carried out in conjunction with the Committee at the Building Research Station, nine young engineers from various organizations have spent at the Station a period of study ranging from 2 to 6 weeks, in order to make themselves familiar with the basic principles of soil-mechanics and with the general methods of soil-examination. Most of these men are normally engaged in supervising practical engineering works, and the contacts thus established should provide a basis for valuable future co-operation between the research workers and the practising engineers concerned in the development of the subject. As far as can be ascertained, five organizations have already formed soil-testing laboratories and have installed apparatus of designs similar to those in use at the Building Research Station and at the Road Research Laboratory. In addition, a number of universities now have apparatus for carrying out research into soil-mechanics problems.

Since much of the information on soil-mechanics is not readily accessible to engineers in Great Britain, the Building Research Station has been engaged in the preparation of a report describing modern methods of analysing problems relating to foundations and earthworks. The prepara-

[1] Journal Inst.C.E. vol 12 (1938–39), p. 106 (June 1939).

tion of the report, which is intended to serve as a convenient introduction to the subject for engineers, is being pushed forward as time and circumstances permit.

Other work at the Building Research Station, which has been undertaken in part for the Road Research Laboratory where work has been proceeding on allied problems of more direct application to roads, has been concerned with investigations in connexion with specific practical problems. Sixteen investigations have been carried out to study the soil-properties and site conditions at sites where engineering works were either contemplated or where difficulties had been encountered during construction. In the course of these investigations valuable data and experience on a variety of problems have been gained, and in many cases the co-operation of the engineers, which has been very readily forthcoming, in carrying out tests and observations on the site, should add appreciably to the potential value of the information collected. To illustrate this point reference may perhaps be made to one or two of the major investigations which have been carried out.

One investigation involved the examination of a fairly extensive site at which the subsoil consisted of a very deep deposit of soft alluvial soils. The object of the investigation was to provide data to assist the engineers in deciding upon the most suitable form of foundation for the buildings to be erected on the site. The information required concerned the degree of variation in the soils over the site, and a quantitative estimation of those soil-characteristics from which the foundation behaviour of the soil can be ascertained. To obtain the required data laboratory-tests were carried out on a large number of soil-samples from various parts of the site, and from different depths ranging from the surface layers to as much as 128 feet below ground-level. The results obtained are, of course, valuable from the research point of view for record purposes, and, in this, case their value was increased by the fact that the contractor's engineers also carried out on the site loading tests on areas up to 4 feet square, and pile-loading and pile-pulling tests. The correlation between field-tests and laboratory-tests which was thus made possible should prove valuable for future investigations.

In another case the design and construction of an anchorage in a tidal river gave rise to a number of interesting problems in the field of soil-mechanics. Among these may be mentioned :

(*a*) the stability and settlement of the embankments ;

(*b*) the suitability of the soil for pumping in hydraulic-fill operations, and the settlement of the hydraulic fill ;

and (*c*) the angle of slope at which the sides of the dredged basin would be stable.

Again, especially as the site was of an extensive nature, a large number of samples were examined to explore the degree of variation in soils over the site and to facilitate the selection of representative soil-types. A

comprehensive investigation of the properties of selected samples, including the determination of shear- and consolidation-characteristics, was then made, and yielded an amount of valuable data. In addition, the results of this work suggested the desirability of tests in the field to record the pressure in the pore water of the foundation soil during the construction of the embankments; the resident engineer undertook these observations, which should not only be useful in assessing the safe rate of construction of the embankments, but should also provide a means for checking the theoretical methods of analysis.

Another investigation related to the settlement-analysis of an important new bridge which is being constructed on a thick stratum of clay. For this purpose the subsoil was explored by boring to a depth of some 40 feet beneath each of the piers and the abutments, and a large number of soil-samples were obtained and submitted to laboratory examination. A detailed knowledge of the properties of the clay deposit and a fairly complete picture of the degree of variation within the deposit was thus obtained, which has a value quite apart from the immediate construction because of the importance of the soil-stratum in question. In addition, however, the quantitative experimental values of the various soil-characteristics permit an estimate to be made, on the basis of current theories, of the probable settlement of the various footings of the bridge. The actual conditions obtaining in the practical case make the settlement-analysis somewhat complicated, so that arrangements that have been made possible through the co-operation of the engineer for settlement readings to be made both during and after construction, are particularly important as a check.

The sites concerned in these cases are frequently of an extensive nature, and since soil-deposits are rarely, if ever, homogeneous, the need for a means of estimating the degree of variation in soils over the site has been emphasized. The laboratory-tests which are required to determine quantitative values of the important soil-characteristics involve a somewhat elaborate testing technique, which means that on the grounds of cost such tests must be limited to those samples which represent the most frequent and important soil-types. To select these representative samples it is necessary to examine a sufficiently large number of samples to determine accurately the variation in soil-type and soil-condition in both a vertical and a horizontal direction over the whole site. The examination, while being simple, rapid, and comparatively inexpensive, should, however, be capable of giving a quantitative indication of the degree of variation encountered. For this purpose the Building Research Station has developed a comparatively simple technique, which may be briefly described as follows. Small cylindrical samples of "undisturbed" soil, about 1½ inch diameter and 5 inches long, are obtained by means of a small sampling tool, used in conjunction with a post-hole auger fitted with extension rods of ¾-inch gas-barrel. This apparatus is readily portable

and is quite efficient in soft alluvial soils and in clay soils. The samples are then tested in the field for their compressive strength by means of a portable compression apparatus developed at the Station, and described elsewhere[1]. The apparatus provides an autographic record of the load-deformation curve up to failure, which means that many samples can be tested in a short time by leaving the records to be evaluated at a later date. After test the sample is placed in an air-tight bottle for transport to the laboratory. In this way a rapid survey of a site can be readily carried through. In the laboratory the "natural" moisture-content of the sample is determined by weighing and drying. In addition, simple tests are carried out to determine the "liquid limit" and "plastic limit", which give a good indication of the composition of the soil. Thus a quantitative indication is obtained of both the soil-composition and of the condition at which it exists in the ground. Comparisons between different samples then permit the degree of variation to be ascertained, and indicate the positions from which representative samples for detailed examination may be taken.

In concluding this brief note, the following observations may be made concerning the position of soil-mechanics in Great Britain: firstly, that, in the development of research, close co-operation is being maintained between the soils-laboratory and the practising engineer; and secondly, that in the civil-engineering profession in general there is a developing interest in this new branch of science and in the possibilities presented by its application to practical engineering problems.

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[1] L. F. Cooling and H. Q. Golder, "Portable Apparatus for Compression Tests on Clay Soils." *Engineering*, vol 149 (1940), p. 57 (19 January).

OBITUARY.

CHARLES JOHN BROWN, C.B.E., son of Dr. W. E. Wilkie Brown, was born at The Manse, Bannockburn, Scotland, on the 28th January, 1872, and died at Guildford on the 17th November, 1939. He received his early training at Edinburgh University and the Heriot-Watt College, Edinburgh, during the years 1888 to 1893, whilst during the years 1889 to 1893 he was a pupil under Mr. J. B. Young, of the Engineer's Department, North British Railway. From 1893 to 1898 he received further training as an assistant to Mr. James Bell, M. Inst. C.E., the Chief Engineer of the same railway company. On completion of this training he became Assistant Engineer, which post he held until 1909. During that time he designed and supervised many of the major alterations made on the North British Railway. In 1909 he was promoted to Chief Engineer and later, in 1911, he was appointed Chief Engineer of the Great Northern Railway.

After the amalgamation of 1923, Mr. Brown took charge, in addition to his other duties, of the Engineers' Department of the former Great Central Railway and, in 1925, of the former Great Eastern Railway, thus becoming responsible for the whole of the Southern Area of the London and North Eastern Railway. He was a lieutenant-colonel in the Engineer and Railway Staff Corps of the Territorial Army, but resigned his commission in 1937 when he retired from railway service.

Mr. Brown was elected an Associate Member of The Institution in 1900, and was transferred to the class of Member in 1905.

In 1901 he married Alice, daughter of Mr. J. J. Milridge of Edinburgh, and had two daughters.

COLONEL ROOKES EVELYN BELL CROMPTON, C.B., F.R.S., was born on the 31st May, 1845, at Sion Hill, Yorkshire, and died on the 15th February, 1940, at Azerley Chase, Ripon. In 1855 he went to Gibraltar with his father, who took out the Second West Yorkshire Militia, and in the following year he was serving in the Crimea as a naval cadet in H.M.S. "Dragon Fly", receiving the Crimean medal and the Sebastopol clasp. On returning to England he attended Harrow school, which he left in 1860. He was gazetted as an ensign in the Rifle Brigade in 1864, and proceeded to India, being later seconded for special service as superintendent of the Government Steam Train Department.

He was a pioneer in both mechanical road traction and electrical engineering, and was in charge of the experiments which the Indian

government conducted to test the possibilities of motor transport on the great trunk roads. He retired from the army in 1876. On his return to England he devoted his energies to electrical engineering, and in 1878 founded the firm of Crompton & Co. at Chelmsford, with which he retained his connexion for 25 years. The firm pioneered in the field of electric lighting, and one of the first buildings to be lighted in this way was the Royal Courts of Justice, London. The fire which destroyed the Ring Theatre at Vienna, and the subsequent reconstruction of that building gave Crompton the chance to experiment with the distribution of electrical energy from a central station a mile away, 440-volt current being transmitted over a five-wire system. This successful installation was followed by others in various continental cities. In Great Britain Crompton founded the Kensington Court Electric Supply Company, which, in 1886, delivered current at 220 volts over a three-wire system of distribution.

He took an active interest in the scheme for utilizing trained electrical engineers for national defence. In 1900 he went to South Africa in charge of a contingent of the Corps of Electrical Engineers; for his services he was mentioned in dispatches, was awarded the Queen's medal with three clasps, and was made a Companion of the Order of the Bath. One result of his service in South Africa was to stimulate his interest in road transport by mechanical means, whilst he also devoted considerable attention to the problem of road surfaces. On the formation of the Road Board in 1910 he was appointed Engineer, and in that capacity carried out many investigations on the wearing qualities of surfaces and the reduction of dust. For many years he held a commission in the London Electrical Engineers (Territorial Army), and commanded the unit from 1901 to 1910, when he became Honorary Colonel. During the great war of 1914–18 he was again commissioned, and was actively concerned in the development and design of the first tanks.

Another of his many interests was the problem of standardization. He was one of the first representatives nominated by the Institution of Electrical Engineers to the main committee of the Engineering Standards Committee on its foundation in 1901, whilst he also organized the International Electrotechnical Commission, the first plenary meeting of which was held in London in 1906, with Lord Kelvin as President and Crompton as Honorary Secretary. He also did much work in improving the shape and fit of standard screw threads, and co-operated in the design of the British standard fine thread, to take the place of the Whitworth thread on motor-cars and electrical plant.

Colonel Crompton was elected a Member of The Institution in May 1886, and an Honorary Member in March 1934; he served on the Council from November 1902, to November 1934. In 1891 he presented a Paper on "The Cost of the Generation and Distribution of Electrical Energy"[1],

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[1] Minutes of Proceedings Inst. C.E., vol. cvi (1890–91, Part IV), p. 2.

for which he was awarded a Telford medal and a Telford premium. In 1905 he delivered the James Forrest Lecture on the subject of "Unsolved Problems in Electrical Engineering"[1]. He joined the Institution of Mechanical Engineers in 1877, and became an Associate of the Institution of Electrical Engineers in 1880, being transferred to the class of Member in 1881; he served as President in 1895 and 1908, and was elected an Honorary Member in 1923. The Faraday medal was awarded to him in 1926. He was elected a Fellow of the Royal Society in 1933. He was the first President of the Institution of Automobile Engineers on its foundation in 1906, and was elected an Honorary Member in 1924. He was also sometime President of the Institution of Highway Engineers, of the Commercial Motor Users' Association, and of the Junior Institution of Engineers, whilst in 1903 he presided over Section G of the British Association. He also served on the Executive Committee of the National Physical Laboratory. In 1935, on the occasion of his 90th birthday, he was the guest of honour at a banquet given by the Presidents of the Institutions of Civil, Electrical, and Mechanical Engineers and the International Electrotechnical Commission.

In 1881 Colonel Crompton married Elizabeth Gertrude Clarke, of Tanfield, who died on 27 November, 1939. There were two sons and three daughters of the marriage.

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[1] *Ibid*, vol.clxii (1904–05, Part IV), p. 339.

GRIFFITH JOHN GRIFFITHS was born on the 4th January, 1873, and died at Tunbridge Wells, Kent, on the 16th January, 1940. He was educated at Plymouth College and at Wickwar Grammar School, and received his practical training with Messrs. T. A. Walker & Company, contractors, being engaged on the Warrington division of the Manchester Ship Canal. In 1891 he was appointed to the staff of the Manchester Ship Canal Company, and in 1895 became resident engineer on the Eastham division of the canal. In 1899 he was employed by Messrs. Topham, Jones, & Railton as sectional engineer on the extension of H.M. dockyard, Gibraltar, including the construction of three dry docks for battleships. In 1902 he was appointed resident engineer for the Thames Conservancy, to supervise the maintenance of river channels, locks, etc., from Cricklade to Teddington—a distance of 136¼ miles—and the reconstruction of ten locks and two large weirs. In 1910 he became chief engineer to the Conservancy Board, being responsible for the maintenance of all channels and works under the Conservators' jurisdiction from Cricklade to Teddington, including the reconstruction of sixteen locks and forty weirs, the construction of two mechanical boat-conveyors, the construction of a dry

dock and wharves, and river-improvement works between Shepperton and Teddington. After his retirement in 1938 he practised as a consulting engineer.

He was a Captain in the Royal Artillery (Territorial) and served overseas in 1916–17 with the 293rd Brigade as Staff Captain and acting Brigade Major in the 58th Divisional Artillery. In 1917–18 he was seconded to the Corps of Royal Engineers, and was Officer in charge of construction at the National Shipyards, Chepstow, with the rank of Major and Deputy Director. In 1919 he was demobilized with the rank of Captain from the Royal Artillery (Territorial Army).

He was elected a Member of The Institution in February 1914, and became a member of Council in 1937. He was also a Member of the Institution of Water Engineers. His public services included membership of Government committees on Thames floods in the County of London (1928); gaugings of rivers and underground water (1930); the Departmental Committee on Thames Flood Prevention (1931); and the Inland Water Survey Committee (from 1935).

In April 1902 he married Edith Margaret Dempsey, daughter of Charles John Bushell, of Bromborough, Cheshire. He leaves two sons and one daughter; his younger son is a Captain in the Royal Artillery, with the B.E.F.

SUMMERS HUNTER, C.B.E., was born on the 12th July, 1856, at Inverness, and died on the 7th January, 1940, at his home in Stocksfield, Northumberland. He received his early education at the Royal Academy, Inverness, and at the age of 14 was apprenticed to Messrs. Barker & Cope, at Kidsgrove, near Stoke-on-Trent. He pursued his technical studies in the science classes of the Wedgwood Institute, Burslem, and during his training was placed in charge of various important contracts. In 1880 he joined the North Eastern Marine Engineering Company, Ltd., and after serving for a short time in the shops he went to sea for 2 years, taking his chief engineer's certificate. He was then appointed to the staff of the company, of which he became general manager in 1895. He was elected a director in 1899, became managing director in 1900, and was appointed chairman in 1920. He retained these offices until his retirement in 1929.

Mr. Hunter was associated with several advances in marine engineering, including superheaters, the use of special metals in marine engines, the manufacture of marine oil engines, and the fitting of motor-ships with an exhaust-turbo-charged installation. He early recognized the possibilities of electric drive, and effected the entire electrification of the company's Wallsend works in 1900, whilst in the following year the Sunderland works were also electrified.

During the last war he served on various Government committees, including the Board of Trade Shipping and Shipbuilding Committee, the Shipping Controller's Committee, the North East Coast Armaments Committee, and the Tyne and Wear Board of Management of the Ministry of Munitions. In recognition of his services he was awarded the C.B.E. in 1918. He also took an active part in the Territorial Army movement, being instrumental in forming two companies in Wallsend, and held the rank of Captain in the Northumberland Fusiliers.

He was elected a Member of The Institution in February 1910, and served on the Council from 1914 to 1922. He was President of the North East Coast Institution of Engineers and Shipbuilders from 1908 to 1910, and was honoured by that Institution with its Honorary Fellowship in 1932. He was also President of the Institute of Marine Engineers in 1912; a Vice-President of the Institution of Naval Architects; and a member of the Institution of Mechanical Engineers, the Institute of Metals, and the Royal Society of Arts. His activities also included the chairmanship for some years of the Standing Committee of the North East Coast Engineering Employers' Federation, membership of the Council of Armstrong College, Newcastle-on-Tyne, and service on the board of the Wallsend Urban District Council.

In 1887 he married Dora Elizabeth Elliott, of South Shields, who died in 1936, and is survived by four sons and three daughters.

SIR JOHN RUMNEY NICHOLSON, C.M.G., was born at Langwathby, Cumberland, on the 25th March, 1866, and died on the 22nd November, 1939, at his residence at Keswick. He was educated at St. Bees School, and commenced his career with Messrs. Black, Hawthorn & Company, at Gateshead-on-Tyne. From 1888 to 1889 he was in charge of the erection of the Pangdon Dene power-station of the Newcastle-on-Tyne Electric Supply Company. In 1889 he was appointed assistant engineer of the Quebrada Railway and Copper Mines, Venezuela, and in 1891 became chief mechanical engineer of this undertaking, and also of the South Western Railway of Venezuela. From 1895 to 1899 he was associated with Messrs. P. W. and C. S. Meik, consulting engineers, and during this period he designed the locomotives, rolling stock, and other plant for the Port Talbot Railway and Docks, and was resident engineer of the graving docks at Port Talbot. From 1899 to 1902 he was chief engineer of the Bridgwater Trust, until his appointment as managing director and chief engineer of the Tanjong Pagar Dock Company, Singapore. When this company's property was acquired by the government in 1904, he became chairman and chief engineer of the Singapore Harbour Board and of the Penang Harbour Board. After the completion of the King's dock, Singapore, he

was awarded the honour of C.M.G. in 1913, and was created a Knight Bachelor in 1919 upon the completion of the reconstruction of the Singapore dock scheme. He returned to England in that year to occupy the position of chief engineer for docks of the North Eastern Railway, and when the railways were amalgamated in 1923 he was appointed to a similar position with the London and North Eastern Railway. He retired in 1927, but at the request of the directors of the railway, acted as consulting engineer for docks for a further 2 years.

Sir John was elected an Associate Member of The Institution in April 1897, and was transferred to the class of Member in February 1903.

In 1902 he married Sybil Helen Croft, O.B.E., daughter of the late Sir Herbert Croft, Bart., who survives him, and had one son and one daughter.

ABSTRACTS OF THE CURRENT TECHNICAL LITERATURE OF ENGINEERING AND APPLIED SCIENCE.

ENGINEERING CONSTRUCTION.

A Projection Method of Mapping from Air Photographs. H. G. FOURCADE (**Trans. Roy. Soc. S. Afr.*, 27, *321–367 ; Jan. 1940*).—In 1926 (*Engng. Abstracts, Part* 32, No. 1 ; *July 1927*) the Author described the stereogoniometer, an instrument for the determination of the relative tilts in space of pairs of photographs taken from the air without ground control, or of their absolute tilts given ground control. He now describes means whereby both images, which may have been photographed at different heights, remain in sharp focus at all projection-levels, whilst their geometrical relations remain mathematically accurate, and at the same time a wide range of scales for the constructed maps is made available. Each picture is rectified to a horizontal plane and constant principal distance, and the rectified pictures are then projected, in an instrument designated the stereoprojector, to a common plane and any required scale within limits. This instrument is described, and its applications are explained in detail.

A Photographic Study of Fluid Flow between Banks of Tubes. R. P. WALLIS (**J. & Proc. Inst. Mech. Engrs.*, 142, *379–383 ; Feb. 1940*).—The Author describes investigations made at the Imperial College, London, in which photographs were taken of moving specks of aluminium dust sprinkled on the surface of flowing water. Details are given of the water-speeds, illumination, camera exposure, and other relevant conditions, and the effects of transverse and longitudinal pitch in both parallel and staggered nests of tubes are illustrated by photographs.

Sealing the Lagoon Lining at Treasure Island (San Francisco Bay) with Salt. C. H. LEE (**Proc. Amer. Soc. Civ. Engrs.*, 66, *247–273 ; Feb. 1940*).—The Author describes the method by which the 10-inch clay lining of the bottom of the 7-acre fresh-water lagoon for the Golden Gate International Exposition was sealed cheaply by a priming of salt water pumped in from the bay. The lining was compacted by a 14-ton flat roller. The initial seepage loss from fresh water in a test-pool was 1 inch per day ; this was reduced to 0·1 inch per day by the treatment described. The Author states that sealing by other methods, for example, the application

of bentonite, would have cost at least $15,000, and would have involved delay in completing the landscape features of the lagoon. This method of sealing should have a wide field of usefulness in constructing impervious membranes for reservoirs and dams.

Reinforced-Concrete Piles in Foundations. P. G. O'Rourke (**Bull. Inst. Civ. Engrs. Ireland, 1939–40, 41–65 ; Feb. 1940*).—The Author classifies concrete piles under three main heads, namely, pre-cast driven piles, cast-in-situ piles, and bored piles. He discusses site tests, and describes apparatus devised by himself for this purpose. He also discusses the question of reinforcement, the determination of the "set" of a pile, and head-stresses on piles, tabulating test results for typical combinations of pile-section and hammer. Another Table indicates, for various pile-lengths, the safe load, the weight of the hammer, the height of fall, and the set.

Rubber Water-Stops for Dams. C. P. Vetter (**Engng. News-Rec.*, 124, *159–161 ; 1 Feb. 1940*).—After several years' investigation, engineers of the United States Bureau of Reclamation have developed a rubber water-stop, which is in use in the joints of the Imperial dam, Arizona, and on structures of the All-American canal. The Author describes the rubber seals in detail, and gives the results of tests.

Water-Conservation and Flood-Protection in Queensland. W. H. R. Nimmo (**Commonw. Engr.*, 27, *197–203 ; 1 Jan. 1940*).—The Author describes the Somerset dam, situated on the Stanley river, 4½ miles above its confluence with the Brisbane river, and about 38 miles north-west of Brisbane, which is being constructed for the water-supply to the metropolitan area and to mitigate floods in the Brisbane river valley. The dam is of gravity type, straight in plan, and is approximately 1,000 feet in length, including a central spillway section of 250 feet controlled by eight sector gates. When completed the structure will contain about 289,000 cubic yards of concrete. The estimated cost is £1,875,000.

Design of Slabs in Liquid-Containing Structures. C. E. Reynolds (**Concrete Constr. Engng.*, 35, *63–72 ; Feb. 1940*).—The Author's discussion applies to slabs forming the walls or floors of tanks or reservoirs designed in accordance with the stresses recommended in the code of practice issued by The Institution[1]. He considers three practical cases, namely, slabs subject to bending only (as in cantilevered walls) ; slabs subject to direct tension only (as in the walls of cylindrical containers) ; and slabs subject to direct tension and bending (as in the walls of rectangular containers).

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[1] "Recommendations for a Code of Practice for the Design and Construction of Reinforced-Concrete Structures for the Storage of Liquids." The Inst. C.E., 1938.

The Lake Washington Pontoon Bridge, Seattle. C. S. ANDREWS (**Civ. Engng., N.Y.*, 9, *703–706 ;* 10, *17–20 ; Dec. 1939 and Jan. 1940*).—The Author presents a general review of the project and discusses a number of design problems. The floating structure, consisting of cellular reinforced-concrete flat-bottomed boats 59 feet in overall width and ranging from 117 feet to 378 feet in length, will extend over the lake for a distance of 6,560 feet. A floating movable pontoon will afford a 200-foot clear waterway for large ships, whilst a 200-foot opening with 30 feet clearance will be provided near each shore under the fixed steel spans, for pleasure craft.

In the second article the Author describes the method of constructing and launching the pontoons, the setting of the heavy anchors holding the structure, the adjustment of cable-lengths, and the provisions for rigidly connecting adjacent pontoons. The bridge is expected to be completed by July, 1940.

Redundant Frames. C. N. ROSS (**J. Instn. Engrs. Australia*, 11, *421–424 ; Dec. 1939*).—A description is given of a modification of existing methods of determining the forces in the members of a redundant frame. By the use of trigonometrical resolution, or other standard method, the forces in all members are written on the frame-diagram ; then each redundant bay in the structure is considered by itself, and from the strains on the six members of the bay an equation is derived which contains some of the unknown forces. Two worked examples are given.

Further Investigation of the Creep or Flow of Concrete under Load. W. H. GLANVILLE and F. G. THOMAS (**Dept. Sci. Ind. Research, Building Research Tech. P. No. 21, 44 pp.* ; *1939*).—The Authors present the results of prolonged loading tests on small cylinders of plain concrete and on reinforced-concrete columns. The investigation included creep in pure tension, lateral movements under compression, and the effect of creep upon the deformation and the ultimate strength of reinforced-concrete beams. Measurements of the deformations of a reinforced-concrete arch in a London hall over a period of 8 years are recorded, and an analysis is made of these measurements.

Design of Earthquake-Resisting Buildings. R. E. DE SÁ and G. SINGH (**Quarterly Technical Bulletin* (*Railway Board of India*), 5, *15–29 ; Jan. 1940*).—The Authors give rules for the design of masonry and framed buildings, which have been used on the North Western Railway of India since 1931. They present a mathematical analysis of the stresses involved under earthquake conditions, and include an example of design.

The Arkalon Cut-off on the Chicago, Rock Island and Pacific Railway. (**Rly. Age, Chicago*, 108, *202–207 ; 223 ; 27 Jan. 1940*.)—The Arkalon cut-off, near Liberal, Kansas, replaces a 12-mile section of line in the deep

Cimarron valley, constructed on alternate cuts and fills from 12 feet to 15 feet high and from 200 feet to 1,200 feet in length, with sharp curves and heavy gradients. The work involved the construction of 8·42 miles of new line on fill sections up to 95 feet high, about 3 million cubic yards of grading, and the erection of a five-span high-level bridge, 1,269 feet in length, consisting of five deck-truss spans each of 250 feet, with a top-chord elevation 97 feet above the river-bed, supported on reinforced-concrete piers and abutments. Special provisions were made to protect the bridge against fire and floods.

Remote Control of Facing Points at Running Loops on the French National Railways. EPINAY (**Rev. Gén. Chem. de F.*, 58*ii*, *277–295*; *1 Dec. 1939*).—A description is given of the method of controlling the facing points leading into running loops between Etampes and Orleans, where automatic colour-light signalling is installed. Each set of points is normally operated from the signal-cabin controlling the exit from the loop, by means of a small panel with relay interlocking. The knob controlling the points can be in any of three positions: turned to the left, the points are set for the main line, and the covering signals work automatically; in the central position, the junction signal is maintained at danger; and turned to the right, the points are set for the loop and the signals work automatically, but display a board showing the limiting speed (70 kilometres (44 miles) per hour). Since relay interlocking is provided, the knob may be reversed completely while a train is travelling over the approach-track and points, which will operate only when the train is clear of them. To avoid any risk of vehicles with a bad shunt causing the track-circuit relays to clear, and thus to allow the points to move under the train, the current from the control-knob is transient. Hence if the points do not move when the knob is turned, the latter must be restored to the mid-position, and again turned to the desired position. Provision is made for local operation of the points when necessary.

Signalling Arrangements for Rail Traffic over a Swing Bridge in Bengal. M. A. FARUQUI (**Quarterly Technical Bulletin* (*Railway Board of India*), 5, *5–6*; *Jan. 1940*).—The bridge, on the Sara-Sirajganj line, carries a single track. The bridge-bolts are worked from a single-lever ground-frame fitted with a key lock, and are connected with a vertical plunger in line with the bridge race-rail. The ground-frame lever is therefore prevented from being pulled to the "bridge-locked" position except when the bridge is correctly lined-up for rail traffic, as only in that position is there a notch in the race-rail opposite the vertical plunger. When the bridge is locked the withdrawal of the key back-locks the ground-frame lever. The key may then be used to unlock either of the home signals. Each home signal is worked by rodding from a double-wire-type lever mounted on the post, and the lever simultaneously operates

the outer signal. Operation of the lever back-locks the key, so that in addition to ensuring that the bridge is locked before the signals are cleared, the arrangement prevents simultaneous clearing of the "up" and "down" signals.

A New System of Light Signals on the Canals of the Venetian Lagoon. D. Alessi (**Ann. Lav. Pubbl.*, 77, *1136–1140 ; Nov. 1939*).—The Author gives a detailed description of the system of lighting the navigable canals between the basin of San Marco and the Lido, for the guidance of traffic at night and in fog. This consists of rows of electric lights on standards formed of three or more piles, shielded so that no direct light strikes the eyes of navigators whilst the standards are brightly illuminated. The routes thus lighted amount to 3·5 kilometres. The total cost of the installation, including the transformers and other apparatus, was 350,000 *lire*, and the annual cost of maintenance is given as 15,000 *lire*.

Sea-Defence and River Works in East Norfolk. K. E. Cotton (**Civ. Engng., Lond.*, 35, *6–10 ; Jan. 1940*).—A detailed description is given of the works carried out to close the breach caused by high seas at Horsey in February, 1938, when an area of about 15 square miles was flooded, and of the construction of more permanent protective works, using the temporary bank of concrete-filled bags, with a core of sandbags as a foundation.

Arterial Watermains, with Reference to the Liffey Supply Scheme. D. K. Ryan (**Bull. Instn. Civ. Engrs. Ireland, 1939–1940, No. 1, 29 pp. ; Jan. 1940*).—The works in connexion with the Dublin water-supply include the laying of 13 miles of large water-mains. The Author discusses the preliminary tests made with steel and cast-iron pipes and compares the characteristics of these types. He details the specifications, and tabulates test results. He describes the methods adopted in laying the steel pipe-lines, in order to avoid damage to the bituminous protection of the pipes. In laying the 27-inch cast-iron main through the city, the depth of cover provided was not less than 4 feet at any point where the main was laid under the carriageway, in order to minimize the effects of vibration and impact from traffic, and to avoid undue interference with existing services.

MECHANICAL ENGINEERING.

Coke Breeze and Pitch as Fuels for Boiler-Firing. (**Power & Works Engr.*, 35, *17–21 ; Jan. 1940*).—Efforts to burn coke breeze efficiently have been made for many years. At two works of the Gas Light and Coke

Company, the problem has been solved without the aid of refractory arches. Pitch is not used alone, but is fired in conjunction with coke breeze on chain-grates. Details of the plants are given.

Researches on the Introduction of Aqueous Vapour into the Cylinders of Internal-Combustion Engines. C. DE GREGORIO (**Ricerche Ing.*, 7, *186–195 ; Nov.–Dec. 1939*).—The Author describes experiments made in the mechanical laboratory of the University of Palermo on a two-stroke diesel engine. The results are tabulated and plotted in curves. The conclusions drawn are that a small quantity of vapour may slightly improve the total output of the engine. A large quantity is always harmful; it brings with it a sensible increase in the specific consumption and a general cooling in the cycle. Part of the water introduced always becomes dissociated, with beneficial effects. If the admission of water-vapour is kept within the non-harmful limit, the possible increase in the compression-ratio should be small, since the cooling produced by the water should be nil, or nearly so.

Springs for Large Diesel Engines. C. E. SQUIRE (**Diesel Engine Users Ass., Paper No. S.155, 8 pp.; 1939*).—After reviewing the present state of knowledge in regard to the design of helical springs, the Author presents the results of tests made on a machine having a capacity of 1,000 lb., running normally at 400 revolutions per minute, but occasionally up to 600 revolutions per minute; this machine had an adjustable crank, and as the motion was simple harmonic, there was no serious surge in the coils of the springs with the sizes and speeds adopted. The largest series of tests was made on springs with a mean diameter of 2·62 inches and a wire diameter of 0·4 inch. The Author concludes that in the selection of suitable stress for use in designing diesel-engine valve-springs the safe stress values should correspond to spring pressed home coil to coil rather than to the lower-stressed condition corresponding to valve full open. When possible the safe stress should be less than the fatigue-limit—say 0·7 of that limit.

The Kadenacy Engine. (**Engineering*, 149, *195–197 ; 23 Feb. 1940.*)—Descriptions are given of engines which have recently been constructed and tested to demonstrate the advantages of the Kadenacy principle (*Journal Inst. C.E.*, 13, *262* (*Jan. 1940*)), including a single-cylinder model, designed for general industrial purposes, the results obtained from which are tabulated. Two six-cylinder engines of the two-cycle type, operating on the Kadenacy principle, are also described; the first is an experimental model, whilst the second, which embodies a number of modifications, is the commercial type. Test results are plotted in curves.

High-Capacity " Hydro-Blast " Circuit-Breaker for Central-Station Service. W. F. SKEATS and W. R. SAYLOR (**Elect. Engng., N.Y.*, 59 *(Trans.), 111–114 ; Feb. 1940*).—The increasing demand for oil-less circuit-breakers has resulted in the development of the " hydro-blast " type, which has been constructed for ratings up to 1,500,000 kilovolt-amperes. It is completely interchangeable with the standard H-type oil circuit-breaker, and thus combines the advantages of that type with total absence of inflammable material. The Authors describe in detail the construction and operation of a 500,000-kilovolt-ampere unit for central-station service, and state that several of these have been in service for more than half a year. The results of three-phase tests at 14,500 volts, 50 cycles, are tabulated.

An Investigation into the Occurrence and Causes of Locomotive Tire Failures. C. W. NEWBERRY (**J. & Proc. Instn. Mech. E.*, 142, *289–303 ; Jan. 1940*).—The Author describes tests made to determine the causes of particular failures and also to establish general relationships between effect and cause in regard to tire defects. Individual failures are examined, and experimental work directed toward the improvement of wheel and tire is described. From a statistical review the Author concludes that fatigue is the chief cause of tire failure, and he discusses the factors which influence the development of fatigue failure. He states that the occurrence of fatigue failures has been reduced by a change in tire-boring methods to increase the effective fatigue strength of the tire, and by modifications in design to ensure more uniform stress-distribution.

2–12–4-Type Locomotives for the Trans-Balkan Railway. (**Rly. Gazette, Lond.*, 72, *291–293 ; 1 March 1940*).—A brief description is given of the line, which runs from Tirnovo to Stara-Zagora, in Bulgaria, a distance of 79·5 miles. The ruling gradient is 1 in 40 (with short lengths of 1 in 37), and curves of 820 feet radius occur in certain places. The leading and trailing coupled axles of the special 2–12–4-type tank locomotives employed have 35 millimetres ($1\frac{3}{8}$ inch) and 25 millimetres (1 inch) lateral play respectively, and the other four coupled axles comprise the rigid wheelbase of 4,800 millimetres (15 feet 9 inches) ; the third and fourth pairs of coupled wheels have blind tires. There is a controlling device between the Krauss truck and the leading coupled axle, whilst the pin of the rear bogie is allowed 110 millimetres ($4\frac{5}{16}$ inch) lateral displacement. The leading particulars are as follows : two cylinders, $27\frac{1}{2}$ inches in diameter by $27\frac{1}{2}$ inches stroke ; coupled wheels 4 feet $4\frac{3}{4}$ inches in diameter ; boiler-pressure 227 lb. per square inch ; tractive effort, 58,640 lb. ; weight in working order, 147 tons.

New Beyer-Garratt Locomotives for the Bengal–Nagpur Railway. (**Rly. Gazette, Lond.*, 72, *15–18 ; 5 Jan. 1940.*)—Four new Beyer-Garratt

locomotives of the 4–8–2+2–8–4 type have been recently completed for the Bengal–Nagpur Railway. They are intended for use on a sharply-curved and heavily-graded line serving several collieries. The sharpest curve is of 570 feet radius, and the steepest gradient is 1 in 91. The leading dimensions and particulars of the new locomotives, built to the 5-foot 6-inch gauge, are: cylinders, 20½ inches in diameter by 26 inches stroke; coupled wheels 4 feet 8 inches in diameter; coupled wheelbase 15 feet 6¾ inches; total wheelbase 91 feet 9½ inches; total heating-surface, 4,114 square feet (evaporative heating-surface, 3,453 square feet); boiler-pressure, 210 lb. per square inch; total weight in working order, 230 tons. The tractive effort at 85 per cent. of the boiler-pressure is 69,660 lb.

Southern Pacific 4–8–8–2-Type Cab-Ahead Locomotives. (**Rly. Age, Chicago*, 108, *239–240; 248; 3 Feb. 1940.*)—An additional twenty-eight 4–8–8–2-type cab-ahead oil-burning locomotives were built for the Southern Pacific Lines during 1939, of similar general type to those introduced in 1928. The outstanding change in the latest engines is in the use of spring-pad lubricators to all the axleboxes, installed in the axlebox cellars; descriptions are given of their construction and fitting. The leading particulars of the engines are as follows: cylinders, 24 inches in diameter by 32 inches stroke; coupled wheels 5 feet 3½ inches in diameter; boiler-pressure, 250 lb. per square inch; total heating-surface, 9,084 square feet (6,468 square feet evaporative); rated tractive effort, 124,300 lb. The tender, carried on six-wheel bogies, has a water-capacity of 22,000 (U.S.) gallons, and a fuel-capacity of 6,100 (U.S.) gallons. The engine and tender weigh 525½ short tons in working order.

Diesel-Electric Shunting Locomotives for American Railways. (**Rly. Age, Chicago*, 107, *992–995, 998; 30 Dec. 1939.*)—The Baldwin Locomotive Works has recently completed the first of a series of 1,000-horsepower and 660-horsepower B_0–B_0-type diesel-electric shunting locomotives, each equipped with a single direct-injection diesel engine with eight and six cylinders respectively, direct-coupled to a generator. Axle-hung traction-motors are provided, and the electro-pneumatic control-system provides for operating pairs of motors in series (the two pairs being in parallel), with both full field and shunted field. The larger locomotives have a tractive effort at starting of 72,000 lb., with a factor of adhesion of 3·3; that of the smaller locomotives is 60,000 lb. The engines weigh 120 and 100 short tons respectively. Some locomotives of the larger type have been delivered to the Missouri Pacific and the Atchison, Topeka and Santa Fe Railroads.

Express Electric Locomotives for South America. F. GUILLOT (**Rly. Gazette, Lond.* (*Electric Railway Traction, No. 82*), *2–3; 5 Jan. 1940*).—Four 2–C_0+C_0–2-type electric locomotives have been delivered to the

Paulista Railway, for express service on the Jundiahy–Rincao and Ityrapina–Bauru lines. The locomotives, receiving direct current at 3,000 volts, develop 4,050 horsepower continuously at 50·3 miles per hour, with a tractive effort of 30,000 lb., and are designed for a maximum speed of 90 miles per hour. The total weight is 163 tons, of which 120 tons is available for adhesion.

New Trains for the Liverpool–Southport Electrified Line. (**Rly. Gazette, Lond. (Electric Railway Traction, No. 83), 10–20 ; 2 Feb. 1940*).—A description is given of the new rolling stock constructed for the Liverpool–Southport line, in replacement of that originally built in 1904–1910, when the line was electrified on the 600-volt direct-current third-rail system. The trains are made up of two-coach and three-coach units, the former consisting of motor-coach and trailer, and the latter of motor-coach, trailer, and driving trailer. The bodies, underframes, and bogie-frames are of welded construction. The motor-coaches are each fitted with four nose-suspended traction-motors ; these motors have a 1-hour rating of 235 horsepower, taking 340 amperes at 580 volts, and a continuous rating of 184 horsepower, taking 265 amperes at 580 volts. A fully-loaded five-coach train has an initial acceleration from rest of 1·6 mile per hour per second. Electro-pneumatic control is used.

Individual and Automatic Electrical Drive of Machine-Tools. L. BESNARD (**Rev. Gén. Elect.*, 47, *3–16 ; 43–50 ; 6–13 and 20–27 Jan. 1940*).—The Author discusses the reasons governing the adoption of individual drive of machine-tools, and the various means for securing automatic operation under the most diverse conditions of speed, reversability, and interruption of work. He explains how individual drive can be effected advantageously on old machines, so that it becomes possible, in a workshop equipped with transmission shafting driven by a single motor, to utilize a limited number of machines without recourse to this transmission, and thus to obtain more economical conditions of operation.

The Effect of Hydrogen, Arsenic, Titanium, and Miscellaneous Elements on the Welding of Steel. W. SPRARAGEN and G. E. CLAUSSEN (**J. Amer. Weld. Soc.*, 19 (*Weld. Research Suppl.*), *24–30 ; Jan. 1940*).—The Authors present a review of the literature on this subject published up to the 1st July 1938.

Short-Time Creep Tests on Arc-Welded Low-Carbon Steel. N. F. WARD (**J. Amer. Weld. Soc.*, 19, (*Weld. Research Suppl.*), *14–20, Jan. 1940*).—The tests described were made on 0·1-per-cent. carbon steel in the annealed condition and as welded with the alternating-current arc, using a flux-coated electrode of mild steel. Specimens of the welded steel were subjected to constant load at 500°, 700°, and 900° F. The results

are plotted in curves. The Author concludes that the creep-stress limit as a criterion for design is low for this grade of steel. For high-temperature service its use is not recommended in either the original or the welded condition. For temperatures below 500° F. the steel develops sufficient creep-resistant properties in either the cold-drawn or the welded condition. Under no condition was it found desirable to exceed the recrystallization-temperature for creep tests.

Shear Tests of Plug and Slot Welds. C. E. Loos and F. H. Dill (*J. Amer. Weld. Soc.*, 19, *98–103 ; Feb. 1940*).—The Authors describe the preparation of plug welds, and their test to destruction under a shearing load applied in a universal testing-machine. Similar details are given for slot welds. The conclusions drawn are that plug welds in holes made by any of the common methods—punching, drilling, or gas-cutting—or in countersunk holes, may be stressed in shear at the working stresses ordinarily allowed for fillet welds; the characteristics of slot welds are essentially similar to those of plug welds, and they are not affected by the orientation of the applied shearing stress. The test of plug and fillet welds in the same joint shows that they act together in sustaining the applied load, and that they may be safely designed at the working stresses ordinarily allowed for fillet welds.

Flame-Cutting Non-Ferrous Metals and Non-Metallic Materials, and Oxidation of Metals at Elevated Temperatures. W. Spraragen and G. E. Claussen (*J. Amer. Weld. Soc.*, 19 (*Weld. Research Suppl.*), *51–60 ; Feb. 1940*).—The Authors present a review of the literature of the subject to the 1st July 1938. A bibliography of eighty-six references is included, and suggested research problems are enumerated.

The Fabrication of Large-Diameter Pressure-Vessels. J. F. Bechtle (*J. Amer. Weld. Soc.*, 19, *36–39 ; Jan. 1940*).—The Author defines large-diameter pressure-vessels as those having diameters of more than 10 feet. He describes the handling and supporting of the sections of such vessels for welding, and methods of stress-relieving the finished vessels.

The Static Friction of Lubricated Surfaces. A. Fogg and S. A. Hunwicks (*J. Inst. Petroleum*, 26, *1–18 ; Jan. 1940*).—The object of the investigation described was to obtain information in regard to " boundary " friction and the " boundary lubricating " properties or " oiliness " of various substances. The apparatus used was a modification of the Deeley machine. Observations were made to ascertain the effect of the size of balls, of temperature and load, and of the use of various oils for lubricating the surfaces. The results are given in detail. The investiga-

tion dealt only with surfaces of hard steel, but the work is being extended to include other combinations of metals.

The Influence of Various Lubricants upon the Seizure Characteristics of Hard Steel and Bronze. D. CLAYTON (**Engineering*, 149, *131–135 ; 9 Feb. 1940*).—The Author describes experiments carried out at the National Physical Laboratory on hard steel balls, bronze balls, and combinations of bronze and steel, using first ordinary lubricants of various types and then, in the case of bronze and steel together, the thin liquids petrol and water. The object of the investigation was to provide information on the nature of wear and seizure, but the Author observes that the results are of wider interest, being relevant to metal-cutting, worm and worm-wheel behaviour, and the rubbing of metals in the presence of water, petrol, and other fluids.

The Quadrature and Quadraflux Tachometers. E. B. BROWN (**J. Instn. Engrs. Australia*, 11, *417–419 ; Dec. 1939*).—The Author describes two new forms of electrical tachometers, in both of which a revolving-field magneto generator is used in conjunction with an iron-cored indicating instrument so that there are no moving contacts in the electrical circuit. In the first instrument the generator and indicator are combined in a unit, but in the second they may be separated by any desired distance. The latter is especially advantageous for indicating propeller-speeds in large multi-engined aircraft, whilst by means of a change-over switch it may also be used as an indicator to show the exact synchronism of a number of engines. The indicators of both instruments are practically unaffected by variations in temperature.

Remote Supervisory Control on the Wirral Railway. (**Engineer, Lond.*, 169, *102–104 ; 126–127 ; 2 and 9 Feb. 1940*).—By means of the remote photo-telemetering equipment on the Wirral Railway electrification scheme, any of several meter-readings may be selected at the sub-stations, and continuously indicated at the control-room. Particulars are given of the control and indication equipment.

MINING ENGINEERING.

Longwall Working With Induced Breakdown of the Roof Along Rows of Steel Props in Seams of Low Inclination. W. MAEVERT (**Glückauf*, 76, *25–30 ; Jan. 1940*).—The results obtained from trials using rows of steel props are discussed and compared with those obtained with steel or timber chocks, and the limits of applicability of the two methods are considered. The adoption of the new form of support at the rupturing edge of the roof was not found to present any particular

difficulty. In only a few cases was the practicability of the system dependent upon the character of the strata. The introduction of the steel props was followed by a noticeable reduction in the number of accidents. The enhanced safety afforded by rows of steel props has often been doubted, on the assumption that the relative movement of the roof and the floor would cause the props to assume inclined positions and to lose their bearing capacity. Observations by J. Weissner (*Glückauf*, **74**, *370*; **75**, *833*; *1938*) show, however, that the difference between the movements of the roof and the floor in longwall working with rows of props, is actually so small as to be of no consequence as regards the loading and bearing capacity of the props. The difference is limited to 0–2 centimetres, and lies, therefore, within a range which is usually ignored in setting up props at the coal-face. A similar fact was observed at the Sachsen colliery on comparing the positions of the props in the same face working before and after introducing the aligned-props system.

Experiments on Strength of Small Pillars in the Pittsburgh Bed. H. P. Greenwald, H. C. Howarth, and I. Hartmann. (**U.S. Bur.* (*Min. Tech. P. No. 605*, *22 pp.*; *1939*).—Methods of testing seven small pillars in place in the experimental coal mine are described. All pillars were square in horizontal cross-section. The data obtained may be summarized as follows: (1) measurements taken during preparation of one pillar indicated that the coal-bed was under much greater stress than could be accounted for by the thickness of overburden: as erosion has been in progress in this vicinity in recent geologic ages, it appears likely that stress is residual from times when the overburden was heavier; (2) two pillars, tested with the fireclay floor free to move laterally into a surrounding passageway, failed because of flow of the clay from under the pillars at pressures below the strength of the coal itself: flow of the clay pulled the pillars apart at the bottom; (3) three pillars, including the entire height of the bed, were tested with flow of the clay restrained: ratios of lateral dimension to height of these pillars were, respectively, 0·50, 0·75, and 1·00: they failed, respectively, at pressures of 500 lb., 600 lb., and 695 lb. per square inch, these pressures being proportional to the square roots of the ratios; (4) failure of these pillars started at the free surfaces and proceeded inward: there was some development of the double-pyramid form seen commonly in tests of brittle materials; (5) final failure involved a time element evidenced by continued lateral expansion of the pillars under constant load; (6) two pillars comprising, respectively, the upper and lower halves of the coal-bed, failed at pressures of 885 lb. and 920 lb. per square inch; (7) all pillars compressed vertically under load, rapidly under initial loading, and then more slowly as the load increased: vertical compression was a less accurate indication of approaching failure than the continued lateral expansion under constant load; (8) in each test the roof was subjected to the same total load as the pillar, inasmuch

as the hydraulic jacks reacted against the roof: movement of the roof under these loads was less than might be expected, and in no test was the roof damaged.

Mine Pumping of Acid Water. W. Robertson (*Min. Elect. Engr.*, **20**, *239–240 ; Feb. 1940*).—The Author describes the difficulties experienced and the means adopted to overcome the effect of corrosive acid and abrasive material in underground pumping at the Foulshiels and Loganlea collieries. The inner surface of cast-iron valve-chambers was found to be corroded to a depth of about $\frac{3}{8}$ inch, leaving a residue of insoluble combined graphite. In turbine pumps an annular ring had been cut into the impeller-shafts and the feathers were corroded so that the impellers became loose and the shafts were completely ruined. To protect the impeller-shaft it was decided to use a bitumen compound, and this proved effective. After tests of samples of the mine-water the installation of an all-bronze two-stage split-casing pump of the volute type was recommended; this pump had an output of 200 gallons per minute at 250 feet head, when driven at 1,400 revolutions per minute.

Sampling with Drill and Vacuum Collector at the Picacho Gold-Mine, Arizona. L. W. Dupuy (**Engng. & Min. J.*, **141**, *29–31 ; Jan. 1940*).—The Picacho mine has lain idle for 30 years, but is now being worked as the result of experiments with a new drilling and sampling system. Preliminary tests were made with a wagon drill and a 315-cubic-feet-per-minute diesel-driven air-compressor, in conjunction with a vacuum sample-collector equipped with shaker gates and collector-cans. The collector was connected to the drill through suction-hose attached to a closure-hood at the mouth of the drill-hole. The results were so satisfactory that in 1938 a definite drilling and sampling programme was arranged; and sufficient ore was developed during 1939 to justify the installation of a mill.

Mining and Milling Tin-Tungsten Ore at the Mawchi Mine, Burma. J. E. Denyer and K. C. G. Heath (**Bull. Instn. Min. Metall., No. 426, 30 pp. ; March 1940*).—The mine is at an elevation of about 3,600 feet, and the mill at 1,900 feet; the district is of an unusually hilly nature. The lodes outcrop on a steep hillside, and have been developed by adits. Two long crosscuts are used as main haulage-ways, each serving a ropeway bin. There is good natural ventilation, whilst drainage presents no difficulties. The Authors describe in detail the methods of working the mine, and of sampling the ore. The principle adopted in milling is a carefully-controlled reduction in particle-size, with removal of values as soon as they are released from the matrix. The jigs are of the Hartz type. Details are given of the tailings-plant, of the flotation-plant, and of the various ancillary services.

Dust-Count Technique in the Tri-State Mines, U.S.A. H. C. Chellson (**Engng. Min. J.*, 140 (*12*), *29–33 ; Dec. 1939*).—The Author describes the methods used to collect and record dust-count samples during 1939 at mines wherein the free silica-content averages between 90 and 95 per cent. for the district. The apparatus used in the Tri-State mines is a " midget impinger ", which weighs about 10 lb. ; it is self-contained, has a vacuum of 12 inches of water, and the sampling-rate is 0·1 cubic foot of air per minute. The results obtained are tabulated. The Author states that the task of " policing " the mine air by scientific analysis, and the use of modern drilling and ventilation equipment, have contributed a gratifying measure of success in the battle against silicosis.

Air-Conditioning Plant at the Ooregum Mine, Kolar Gold-Field, Mysore. J. Spalding and T. W. Parker (**Bull. Instn. Min. Metall.*, *No. 425, 62 pp. ; Feb. 1940*).—The present bottom level of the Ooregum mine is at 8,330 feet vertically below the surface. At a vertical depth of 8,000 feet the initial rock-temperature is 134° F., and the temperature increases approximately, 9° F. for each additional 1,000 feet of depth. The mean annual surface temperature is 75·6° F. dry bulb, and 65·7° wet bulb, with a mean daily temperature-range of 19·8° dry bulb, and 5·3° wet bulb. About 220 gallons of water per minute is pumped from the upper levels of the mine, but the deep levels are quite dry, and every effort is made to keep the ventilating air as dry as possible. The air-conditioning installation is an ammonia refrigerating-plant to cool 10,000 lb. (150,000 cubic feet) of air per minute from an intake wet-bulb temperature of 730° F. to 40° F. (saturated), equivalent to 13,764,000 B.Th.U. per hour. The ventilation circuit of the mine is also described, and the Author discusses the cooling effect obtained underground by the air-conditioning plant at the surface. He also gives data showing the improvement obtained in the working results of the mine.

Occurrence and Prevention of Mine Fires caused by Spontaneous Ignition of the Coal. P. Cabolet (**Glückauf*, 75, *953–962, 9719–75, 984–987 ; 16, 23, and 30 Dec. 1939*).—Descriptions of mine fires that have occurred in Germany serve as the basis for an examination of the part played by spontaneous combustion, in remnant pillars and other residual bodies of coal, in the causation of such fires. The Author discusses the causes of spontaneous combustion at the coal-face, and concludes that spontaneous combustion may be a consequence of roof-pressure resulting from ground movement. Certain conditions of bedding, dip, geological disturbances, and primary or secondary stresses in adjacent or remote strata may contribute in producing ignition of the coal. Another possible source of fire in the face-working is washery-waste used for stowing. The Author indicates preventive measures, and discusses steps to be taken on detecting a smell of burning or an outbreak of fire. He considers that

since the odour of the exhaust gases of crude-oil locomotives might mask that of incipient coal-combustion, such locomotives should be used only in parts of the mine where there is no likelihood of fire. The importance of ability to anticipate a threatening fire by the detection of even the faintest odour of burning has led the mines in the Zwickau-Olsnitzer area to employ in working districts liable to fires only such supervisory and senior workers as possess a very keen sense of smell. In Saxon miners the faculty of detecting indications of fire is often so highly developed that they can distinguish between different forms of burning by the odour.

Conveying in the Barnsley Seam at Rossington Main Colliery. (**Iron Coal Tr. Rev.*, **140**, *221–222 ; 2 Feb. 1940.*)—The Barnsley seam lies at a depth of about 880 yards, has a thickness of 5 feet 6 inches, and is nearly level. At the face 24-inch flat-face conveyors are used with a covered-in bottom belt. Loading-stations were established in semi-permanent positions about 200–300 yards behind the face at the start, extending to 800 or 1,000 yards generally before moving up. Details of the installation are given, and it is stated that an output of 1,000 tons per day is regularly loaded by five lads and a corporal.

A Portable Ward-Leonard Winder for A.R.P. (**Colliery Guard.*, **160**, *141 ; 26 Jan. 1940.*)—The danger of dislocation of winding-gear owing to the effects of air-raids has led to the design of a small portable electric winder. The equipment is self-contained, and can be quickly set down at any shaft. All that is necessary is to rig up an overhead pulley and attach a cage to the rope, which, with the winder, provides a complete winding-device by which men can be raised or lowered. The winder is essentially a Ward-Leonard electric hoist mounted on four pneumatic wheels, hauled by a lorry containing a 120-horsepower diesel generating set. The two units are coupled electrically by 50 yards of trailing cable with locking-type sockets. The rope-speed is 6 feet per second.

Back-Stays for Use in Mines. W. A. Johnson and P. G. Taigel (**Safety in Mines Res. Bd., Paper No. 103 ; 24 pp. ; 1939*).—The Authors deal with a method of preventing accidents caused by runaway tubs, by attaching to the end of a tub or a set of tubs a strut designated a backstay, jock, bar-hook, or drag. As these backstays sometimes prove ineffective, tests were made on experimental tracks representing the conditions under which the stays are used in practice. The results are tabulated, and conclusions are presented in regard to the method of attachment of the backstay, its length, weight, and cross-section, and the material to be used in its manufacture, 1·5-per-cent. manganese mild steel being recommended for the purpose.

Centralized Control of a Coal-Washing Plant. (**Engineering,* 149, *250 ; 8 March 1939.*)—The new installation at the Castleford collieries includes power and lighting tranformers and high-tension oil-immersed switchgear, as well as contactor control gear operated from remote push-button cubicles, and more than forty motors, with a total output of nearly 300 horsepower. The method of centralized control enables the control-gear to be installed in a separate switch-house away from the washery: open-type contactors can therefore be used, thus facilitating inspection and maintenance. Groups of motors can be controlled from one position, whilst it is only necessary to provide main power-cables to the contactor-board, smaller cables being run thence to the motors, and multi-core cables to the push-button stations. Alternating current at 3,300 volts is received, and is transformed down to 440 volts for power circuits and to 110 volts for lighting circuits.

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MESSAGE FROM THE PRESIDENT
TO
ALL MEMBERS, ASSOCIATE MEMBERS, ASSOCIATES, AND STUDENTS ENGAGED IN NATIONAL SERVICE.

A very large number of those on the Roll of The Institution are serving their country in various capacities. To all these, in whatever sphere they may be employed, The Institution looks with pride and with the confidence that they will exert themselves to the uttermost in this life and death struggle with the enemies of freedom.

Modern warfare depends in almost every one of its activities on the work of the Engineer, and amongst the most important factors in obtaining victory are the skill, the experience, and the energy of those who have been trained in the high standards of our profession.

To those who have the privilege of active service, whether in the Navy, the Army or the Air Force, or who are in training to take their places with the Forces, we would like to send a message of gratitude, an assurance of full support from the Home front, and an exhortation to remember the heroism and devotion to duty shown by their predecessors of our body in the War of 1914–18.

To those whose technical knowledge and skill are needed in the vast organization working on preparation and production for the needs of the fighting services, we would recall that in a national struggle of this magnitude, such work is of no less vital importance than the duty which falls to those engaged in the fighting line. The maintenance of adequate supplies of high standard not only demands continuous effort in design and manufacture to a high degree of accuracy and reliability, but also involves the construction of depots and factories, of transport facilities and housing on an unprecedented scale, and the production of raw materials from every part of the Empire. To some this may mean merely a continuance, at high pressure, of work which has been customary in peace-time, and it requires perhaps some special effort of self-control to realize the vital nature of the work and to be content to regard it as national service, demanding the same spirit of self-discipline and self-sacrifice as the nation expects from those who have the honour to wear the King's uniform. It should be remembered that lives, and ultimate victory, may depend as much on the reliability of material and construction, and therefore on the Engineer's skill and devotion to duty, as on the valour of those on active service.

There are also those of our number who have been assigned to duty in the defence of our civil population and our national home. Here it needs no great imagination, in view of what has happened to the inhabitants of other free countries, to realize the importance of the duty of civil defence. Not only is this an essential part of the nation's War Effort, but it is a duty owed to those at the front so that they may be confident that their families and homes are being given all possible protection from the horrors of war. Those Engineers who are taking part in this great organization may be assured that The Institution recognizes the value of their work, and is confident that they will give a good account of themselves.

In addition to these obvious categories of national service, it must not be forgotten that there are many Engineers engaged on the essential services of the country, such as transport and communications, water-supply, power-supply, and mining, as well as production for home consumption and export, all of which must be carried on efficiently if the nation is to give full backing to the military effort. To all these Engineers the war has brought special difficulties and responsibilities with the need for ever increasing efficiency and devotion to duty.

Lastly, there are those of our younger members and Students who are still undergoing preparation or training for future national service. We would ask them to use the time that is available to concentrate with utmost energy on the acquisition of such knowledge and experience as is essential to enable them to take their places in due course in this great national enterprise.

To all our members, wherever they may be serving or preparing to serve, we send greetings and encouragement, and we ask them to remember that they carry with them the honour and the high traditions of our profession and Institution, and that they are fighting for the preservation of our freedom, which is in greater jeopardy than it has ever been in the history of our country.

This is no struggle for mastery between great empires and rulers, or even between rival creeds or political systems. It is the revolt of free men against slavery, oppression, tyranny, and cruelty, and the fouling of the wells of truth. There could be no higher cause and none that makes a stronger appeal to men of our calling.

Clement D. M. Hindley

NOTE.—*Pages* [1] *to* [12] *can be omitted when the Journal is bound in volume form.*

NOTICES

No. 6, 1939—40

APRIL, 1940.

MEETINGS, SESSION 1939–40.

ORDINARY MEETINGS.

The following subjects will be brought forward for discussion on the dates indicated below :—

Apr. 23. "**Remodelling of the Assiut Barrage, Egypt.**" J. E. Bostock, O.B.E., M. Inst. C.E.

May 7*. "**Cliff-Stabilization Works in London Clay.**" J. Duvivier, B.Sc. (Eng.), M. Inst. C.E.

Brief Abstracts of these Papers appeared on p. [16] of the February Number of the Journal.

* **NOTE :—The date of this meeting has been changed from that given in the February Journal and on the card of meetings.**

JAMES FORREST LECTURE.

The James Forrest Lecture, on "New Materials for Old", will be delivered by Mr. E. V. Appleton, M.A., D.Sc., LL.D., F.R.S., on Tuesday, 28 May, at 5.30 p.m.

ANNUAL GENERAL MEETING.

The Council, acting on the powers conferred upon them, have decided that the Session shall end on the 30th May, and that the Annual General Meeting shall be held on the 11th June at 5.30 p.m.

For this year, a departure will be made from the usual practice of reading at the Annual General Meeting the Annual Report of the Council. It will, instead, be published beforehand in the June Number of the Journal, to be issued at the beginning of that month. The corporate members present at the Annual General Meeting will be asked that the Report may be taken as read, to enable the President to review the work of The Institution during the period covered by the Report and to address the members.

SPECIAL ANNOUNCEMENTS.

THE PRESIDENT AND THE CORPS OF ROYAL ENGINEERS

The following announcement appeared in the Supplement to the London Gazette of the 5th March :—

TERRITORIAL ARMY—ROYAL ENGINEERS

Sir Clement Daniel Maggs Hindley, K.C.I.E., M.A., M. Inst. C.E., M.Inst.T., M.I.E. (Ind.), V.D., to be Honorary Colonel.

MILITARY SERVICE.

EMERGENCY COMMISSIONS IN R.E. TRANSPORTATION.

The War Office invites members of The Institution who have had experience in railway surveying or railway construction and who are between 31 and 40 years of age to submit their qualifications with a view to being granted Emergency Commissions through the Army Officers' Emergency Reserve. Associate Members under 31 years of age may be granted such Commissions in special cases. Members who are desirous of applying and who have obtained the necessary permission to offer their services (where such permission is required) should apply as soon as possible to the Secretary of The Institution for the Army form.

The fact that a member has lodged a form with the Central Register of the Ministry of Labour does not debar him from volunteering in response to this invitation from the War Office. Members who have already lodged applications for registration in the Reserve should not apply for a second form.

ARMY OFFICERS' EMERGENCY RESERVE.

Corporate members are informed that registration in the Army Officers' Emergency Reserve is proceeding so far as Civil Engineers are concerned, and members who desire to apply for registration, and who are free to do so, may obtain the necessary form of application from the Secretary. After the form is completed it should be returned to the Institution, when a certificate of membership will be attached to it and the application will be transmitted to the War Office.

It should be noted that the Army Officers' Emergency Reserve was formed with two objects :—

(*a*) To deal with applications for re-employment and appointment to emergency commissions in His Majesty's Land Forces.

(*b*) To maintain a register of retired officers, ex-officers, and others possessing the appropriate military experience or certain technical or other special qualifications, who wish to give an undertaking to present themselves for military service if and when called upon to do so.

Applications may be received from Corporate Members between the ages of 31 and 60 years, but applicants over 55 years of age will only be accepted provided that they possess special qualifications or experience.

Upon being accepted for registration in the Army Officers' Emergency Reserve, members will be notified of that fact by the War Office.

No guarantee can be given that immediate use will be made of the services of all members of this Reserve. Employment will depend on the duration of the war and the nature of the individual's qualifications.

NATIONAL SERVICE (ARMED FORCES) ACT, 1939.

Students of The Institution who are 20 years of age and who are liable for Service under the National Service (Armed Forces) Act, 1939, must register at a Local Employment Exchange when their age-group is called, and may obtain from the Secretary a form of certificate indicating their connexion with The Institution, which, upon production to the Interviewing Officers when their age-groups are called, will, it is anticipated, assist them in being posted to the ranks of the Corps of Royal Engineers or to a technical unit in which their qualifications can be employed.

An Associate Member or a Student over 23 years of age who is normally engaged in civil engineering, must register at a Local Employment Exchange when his age-group is called, and should designate himself as a "Civil Engineer" or as a "Pupil Civil Engineer" respectively if he wishes to be covered by the Ministry of Labour's "Schedule of Reserved Occupations."

He should obtain beforehand from the Secretary of The Institution a certificate of membership, for production to the Registration Officer.

POSTPONEMENT OF ENLISTMENT.

Students of The Institution who are liable for compulsory service in H.M. Forces and who desire to apply to have their enlistment postponed for the purpose of sitting for an Institution Examination, should apply to the Clerk to the Medical Board for a copy of Form N.S. 13 (application for postponement certificate) when they are called up in their age-group for medical examination and interview. The Secretary is prepared to answer general inquiries in regard to this matter.

CENTRAL REGISTER OF THE MINISTRY OF LABOUR.

Students of The Institution who are over 23 years of age, and who are not serving in or attested for service with H.M. Forces, may apply to the Secretary for forms for registration with the Central Register.

GENERAL ANNOUNCEMENTS.

PROFESSIONAL CONDUCT.

The attention of members is again directed to the following pronouncements of the Council :—

(i) That it will not be considered to be a breach of the By-laws and Regulations governing Professional Conduct for a member to permit his name to appear with illustrations of works with which he has been professionally connected, published as parts of advertisements by contractors or manufacturers, but the member is expected to ensure that his name appears in an unobtrusive manner and not in any way as suggesting solicitation of professional work.

(ii) That the pronouncement of the Council, dated May 1934, has been rescinded, namely :—

That while no objection would be taken to the appearance of the names of Chartered Civil Engineers on memorial stones on completed works it would be considered unprofessional for such names to be exhibited on works in course of construction, except where necessary for the purpose of direction, such as on the door of the resident engineer's office ;

and that the following has been substituted :—

That there is no objection to the appearance of the names of Corporate Members on commemorative tablets and stones on completed works, and that a Corporate Member may exhibit his name on works in course of construction.

BAYLISS PRIZE.

On the results of the October, 1939, Associate Membership Examination (Sections A and B), at home and abroad, the Council have awarded the Bayliss Prize of £15 to Mr. William Eugene Blackmore, of Colchester, a Student of The Institution.

A REVISED CODE OF PRACTICE FOR THE USE OF STRUCTURAL STEEL IN BUILDINGS.

A Joint Committee of the Institution of Civil Engineers and the Institution of Structural Engineers was formed, with the following terms of reference :—

(*a*) To draw up a revised Code of Practice for the use of structural steel in buildings, based on the final report of the Steel Structures Research Committee.

(*b*) The Revised Code when completed to be submitted to the Councils of the Institution of Civil Engineers and the Institution of Structural Engineers for approval and joint publication.

The revised Code has now been approved and published, and members may obtain copies from Messrs. Wm. Clowes & Sons, Ltd., St. Mary's, Ballygate, Beccles, Suffolk, at a charge of 6*d*. each (post free).

THE JOURNAL.

The remaining publication dates of the Journal for Session 1939–40 are the 4th June and the 15th October, 1940.

THE MYDDELTON CUP.

The following letter, dated 13 February, 1940, has been received from the American Society of Civil Engineers :—

> During the recent meetings of the Board of Direction here at Society Headquarters the Myddelton Cup was placed on the Board table in order that all might enjoy looking at such a thing of beauty as it is.
>
> The following is the resolution unanimously adopted by the Board thanking the Institution of Civil Engineers for this gift :
>
>> " WHEREAS, The Institution of Civil Engineers, of London, England, has presented to the American Society of Civil Engineers, a replica of that beautiful specimen of English Seventeenth Century craftsmanship known as the Myddelton Cup, commemorative of the bringing of the first supply of potable water to the City of London, and
>>
>> WHEREAS, the gift has been designated by the Institution as a Token of Esteem,
>>
>> BE IT RESOLVED, that it be gratefully accepted and carefully preserved by the Society as an enduring symbol of the mutual regard and affection these two associations hold, the one for the other."

NOTE.—Brief particulars of this Cup, with a reproduction of a photograph, appeared in the " Notices " section of the February, 1940, Journal (pp. [7] and [8]).

CHANGES OF ADDRESS.

Owing to the number of changes of address incidental to service with H.M. Forces, it is not practicable to register such addresses in the List of Members. It is therefore suggested that a home (private) address be maintained, from which communications issued by The Institution might be re-directed. If, however, this is impracticable, The Institution may, in special circumstances, arrange for the dispatch of the Journal as issued to a service address.

SERVICE IN THE FORCES.

For office purposes, a record is being kept of members' service with H.M. Forces, and members who have not already done so are asked to inform the Secretary of such service, i.e. unit, rank, promotions, decorations, etc.

TRANSFERS, ELECTIONS, AND ADMISSIONS.

Since the 20th February, 1940, the following elections have taken place :

Meeting.	*Member.*	*Associate Members.*
19 March.	1	33

and during the same period the Council have transferred three Associate Members to the class of Members, and have admitted fifty-four Students.

DEATHS AND RESIGNATIONS.

The Council have received, with regret, intimation of the following deaths and resignations :—

Deaths.

Burrows, Thomas Edward. (E. 1902.)	*Member.*
Millican, Charles George. (E. 1918. T. 1934.)	,,
Nelson, George Geoffrey. (E. 1911. T. 1933.)	,,
O'Connor, Richard Francis Mary, B.A., B.E. (E. 1903. T. 1936.)	,,
Ravenshaw, Henry Willock. (E. 1890. T. 1918.)	,,
Vauclain, Samuel Matthews, D.Sc. (E. 1907.)	,,
Gates, Harald Burton. (E. 1907.)	*Associate Member.*
Hinchsliff, Edward Robert. (E. 1908.)	,, ,,
Powell, Ernest Brecon. (E. 1916.)	,, ,,

Resignations.

Alluba, Mahmoud Aly, M.Eng. (E. 1916. T. 1926.)	*Member.*
Campbell, Archibald Sidney, O.B.E. (E. 1917.)	,,
Inagaki, Hyotaro. (E. 1908.)	,,
Ker, William Arthur. (E. 1896. T. 1913.)	,,
Storm, John. (E. 1910.)	,,
Beard, Francis Patrick Barry. (E. 1924.)	*Associate Member.*

RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, British Standard Specifications, etc., are not included.]

Aerodynamics. Bairstow, L. "Applied Aerodynamics." 2nd ed. 1939. Longmans Green. 65*s*.

The general plan of the book is that of the first edition (1919), but about three-quarters of the contents is new material. The growth of aeronautics during the past 50 years is reviewed, and the principles derived from that growth are developed, with numerous illustrative examples. The book aims at giving a substantial account of the methods of treating problems arising from the motion of aircraft.

Air Defence. *See* Civil Defence ; Sandbag-Revetments.

Automobiles. Platt, M. "Automobile Brakes and Brake Testing." 1938. Pitman. 3*s*. 6*d*.

Blueprint Reading. *See* Welding.

Brakes. *See* Automobiles.

Building. *See* Price-Books.

Civil Defence. Glover, C. W. "Civil Defence." 2nd ed. 1940. Chapman and Hall. 36*s*.

This is a comprehensive manual dealing in a practical manner with the problems confronting modern society for the adequate protection of the civil population against aerial attack by high-explosive or gas bombs. Communal and individual shelters are described, with relative costs. Legal responsibilities are described, and the organization of A.R.P. services is detailed, including the principles and application of camouflage.

Coal. D.S.I.R. Fuel Research. Physical and Chemical Survey of the National Coal Resources No. 49. "The Cumberland Coalfield. The Six-quarters Seam." 1940. H.M.S.O. 2*s*.

The six-quarters seam is the lowest generally worked in a Cumberland coalfield.

In carrying out the survey pillar samples were cut from localities including four sites under the sea. The data given include proximate and ultimate analysis, forms of sulphur, contents of phosphorus and chlorine, calorific value, and yields of products on carbonization in the laboratory at 900° C., with analyses of typical specimens of the ash.

CORROSION. *See* WATER-MAINS.

ELECTRICAL UNDERTAKINGS. * GARRETT, F. C., *Ed.* "Garcke's Manual of Electrical Undertakings, 1939/40." 37*s.* 6*d.*

ELECTRICITY METERS. STUBBINGS, G. W. "Electricity Meters and Meter Testing." 1939. Chapman and Hall. 14*s.*

ENGINEERING DESIGN. TAYLOR, J. E., and WRIGLEY, J. S. "Engineering Design." 1939. Pitman. 10*s.* 6*d.*

—— EDUCATION. ENGINEERING COUNCIL FOR PROFESSIONAL DEVELOPMENT. "Present Status and Trends of Engineering Education in the United States." Report by *Prof.* D. C. Jackson. 1939.

FENS (The). DARBY, H. C. "The Draining of the Fens." 1940. Cambridge University Press. 21*s.*

This book describes the exploitation of the fens from the sixteenth century to the year 1900, covering the outfalls, administrative problems, and agricultural consequences of the draining; and in an Epilogue the Author discusses in detail the legacy of difficulties bequeathed to the twentieth century as a result of the work of 250 years.

FLOORS AND FLOORING. DAVIDSON, D. M. J. "Floors and Floorings." 1939. Crosby Lockwood. 3*s.* 6*d.*

This text-book comprises two main sections, the first dealing with structural floors, and the second with floor-coverings. It is intended to provide sufficient data to enable the wisest choice to be made for any particular type of job.

GEAR-CUTTING. COLVIN, F. H., and STANLEY, F. A. "Gear Cutting Practice." 1937. McGraw-Hill. 20*s.*

GRINDING. COLVIN, F. H., and STANLEY, F. A. "Grinding Practice." 1937. McGraw-Hill. 20*s.*

HEAT-ENGINES. RIMMER, A. "Definitions and Formulæ for Students. (Heat-Engines)." 4th ed. 1939. Pitman. 6*d.*

This booklet is intended for students, not only for general reference, but also when revising their knowledge prior to examinations, etc. The definitions are summaries of essential points rather than rigid or complete statements. The symbols and abbreviations are those advocated by the British Standards Institution.

IRON-ORE. RICKMAN, A. F. "Swedish Iron Ore." 1939. Faber. 8*s.* 6*d.*

IRRIGATION. INDIA. CENTRAL BOARD OF IRRIGATION. "Quarterly Bulletin, No. 15 July–Sept. 1939." 1939. Simla.

—— PUNJAB IRRIGATION RESEARCH INSTITUTE. "Report for the Year ending April, 1938." 1939. Superintendent, Government Printing, Lahore.

LAND DRAINAGE AND RECLAMATION. AYRES, Q. C., and SCOATES, D. "Land Drainage and Reclamation." 2nd ed. 1939. McGraw-Hill. 26*s.*

—— DARBY, H. C. "The Draining of the Fens. 1940. Cambridge University Press. 21*s.*

MECHANICS HANDBOOK. CAMM, F. J. "Practical Mechanics Handbook." 2nd ed. 1939. Newnes. 6*s.*

MINERAL INDUSTRY. *ROUSH, G. A., *Ed.* "Mineral Industry during 1938." Vol. 47. 1939. McGraw-Hill. 76*s.*

MODELS. *See* RIVERS.

MUNICIPALITIES. *FORBES, J., *Ed.* "Municipal Year Book and Encyclopædia of Local Government Administration, 1940." 1940. Municipal Journals, Ltd. 35*s.*

NAVAL CONSTRUCTION. *VINTRAS, A., and others. "Corporatisme ancien de Construction Navale en France." 1939. Académie de Marine, Paris. 25*s.*

NEW YORK. *See* WATER-SUPPLY.

OSCILLOGRAPHS. REYNER, J. H. "Cathode-Ray Oscillographs." 1939. Pitman. 8*s.* 6*d.*

PORTS. U.S. WAR DEPT. CORPS OF ENGINEERS. Lake Series No. 9. "Ports of Fairport, Ashtabula, and Conneaut, Ohio." 1939. Supt. of Documents, Washington. 1 dollar.

PRICE-BOOKS. *WALTERS, F. T., *Ed.* "Laxton's Builders' Price Book, 1940." Kelly. 10*s.* 6*d.*

PROBABILITY, Theory of. PLUMMER, H. C. "Probability and Frequency." 1940. Macmillan. 15*s.*

—— JEFFREYS, K. "Theory of Probability." 1939. Clarendon Press. 21*s.*

RADIOLOGY. LOW, K. S. "Metallurgical and Industrial Radiology." 1940. Pitman. 7*s.* 6*d.*

The principal object of this book is the consideration of metallurgical examinations from a practical point of view. It is intended not only for the metallurgist but also for the moulder, welder, or other technician and executive who may meet with radiological reports.

RIVERS. U.S. WAR DEPT. CORPS OF ENGINEERS. "Model Study of Plans for Channel Improvement in the Chain of Rocks Reach, Mississippi River." 1939. Tech. Memo. No. 104–1.

—— "Model Study of Tidal Currents in East River, New York." Tech. Memo. 125–3. U.S. Waterways Experiment Station, Vicksburg, Miss.

—— "History of the Improvement of the Lower Mississippi River for Flood Control and Navigation, 1932–1939." By *Brig. Gen.* H. B. Ferguson. 1940. Mississippi River Commission, Vicksburg. 2 dollars.

SANDBAG-REVETMENTS. MINISTRY OF HOME SECURITY. A.R.P. Dept. "Notes on the Construction, Maintenance, and Replacement of Sandbag Revetments." 1940. H.M.S.O. 2*d.*

SCIENCE. WOLF, *Prof.* A. "A History of Science, Technology, and Philosophy in the 18th Century." 1938. Allen and Unwin. 25*s.*

SHIPS AND SHIPBUILDING. *"Directory of Shipowners, Shipbuilders, and Marine Engineers, 1940." 33 Tothill Street, S.W.1. 20*s.*

SPECTROGRAPHIC ANALYSIS. BRITISH NON-FERROUS METALS RESEARCH ASSOCIATION. "Quantitative Spectrographic Analysis with the Microphotometer. Part 1. A Review of Published Work." By D. M. Smith. Res. Report Series No. 524. 1939. The Association. 2*s.*

This report deals mainly with the nature and control of the variables encountered in the standardization of electrical and optical conditions for routine analysis. Data for the analysis of various alloys are summarized in tabular form, with reference to a comprehensive bibliography.

STEAM-ENGINES. THURSTON, R. H. "A History of the Growth of the Steam Engine." Centennial edition, with a supplementary Chapter by W. N. Barnard. 1939. Oxford University Press. 17*s.* 6*d.*

SUDAN. *MACMICHAEL, *Sir* H. "The Anglo-Egyptian Sudan." 1934. Faber. 15*s.*

SWEDEN. *See* IRON-ORE.

TECHNOLOGY. *See* SCIENCE.

WATER. JAMES, G. V. "Water Treatment." 1940. Technical Press. 30*s*.

This treatise deals in a comprehensive manner with the treatment of water for all purposes, the purification of effluents, and the sterilization, coagulation, filtration, and storage of industrial and domestic water-supplies. A section of the book is concerned with the disposal of domestic sewage.

WATER-MAINS. BRITISH ELECTRICAL AND ALLIED INDUSTRIES RESEARCH ASSOCIATION. Technical Report F/T134. "Critical Résumé on the Corrosion of Metal Pipes, with particular reference to Earthing to Water-Mains." 1940. The Association. 6*s*.

This is the first report on the research on the effects of earthing to water-mains, undertaken at the request of The Institution of Civil Engineers. A critical review is made of the information at present available, with the object of forming a basis for further research in the laboratory and in the field.

WATER-SUPPLY. GEORGIA DEPARTMENT OF PUBLIC HEALTH. "Practical Procedures for the Water Works Operator." 1939. Atlanta.

—— NEW YORK DEPARTMENT OF WATER SUPPLY, GAS, AND ELECTRICITY. "Water Supply of the City of New York." 1939. New York.

—— ROBERTSON, W. A. "Overseas Engineering Practice in relation to Water Supply Activities." 1939. Victoria, State Rivers and Water Supply Commission.

WELDING. LINCOLN ELECTRIC COMPANY. "Simple Blueprint Reading, with particular reference to Welding and Welding Symbols." 1939. The Company, Cleveland, Ohio. 75 cents.

WIRELESS. DOWSETT, H. M. "Handbook of Technical Instruction for Wireless Telegraphists." 6th ed. 1939. Iliffe. 21*s*.

—— REYNER, J. H. "Short Wave Radio." 2nd ed. 1939. Pitman. 10*s*. 6*d*.

—— STRANGER, R. "The Outline of Wireless." 1939. Newnes. 10*s*. 6*d*.

(* *The foregoing books, with the exception of those marked with an asterisk, may be borrowed from the Loan Library.*)

LOCAL ASSOCIATIONS.

MEETINGS.

The following meetings have been arranged :—

North-Western Association.

Apr. 17. Annual General Meeting and "The Northwich By-pass", by T. A. Proctor.

Southern Association.

Apr. 25. "The Quay Widening at Berths 34–36, Southampton Docks", by G. W. Rooke, B.A., B.A.I., Assoc. M. Inst. C.E. (Southampton.)

May 2.—Annual General Meeting and "Some Factors in the Design of Sewage Disposal Works", by H. C. Whitehead, M. Inst. C.E. (Portsmouth.)

REPORTS.

Birmingham and District Association.

On Thursday, 7 March, Professor B. L. Goodlet, M.A., Assoc. M. Inst. C.E., read a Paper on " The Problem of Atmospheric Pollution."

Bristol and District Association.

On Thursday, 14 March, a discussion on " Air Raid Precautions " was opened by Mr. H. M. Webb, M.C., B.Sc., M. Inst. C.E.

Edinburgh and District Association.

On Wednesday, 13 March, Mr. J. G. Macgregor, M. Inst. C.E., read a Paper on " Modern Railway Maintenance."

Northern Ireland Association.

The following meetings have been held :—Monday, 19 February, when Mr. D. F. Wilkin, B.Sc., Stud. Inst. C.E., read a Paper on " Concrete and the Resident Engineer " ; Monday, 11 March, when the Vernon-Harcourt Lecture on " The Construction of Deep-Water Quays," by Mr. A. C. Gardner, M. Inst. C.E., was read by Mr. R. D. Duncan, B.Sc., M. Inst. C.E.

Southern Association.

The following meetings have been held :—Thursday, 15 February, at Portsmouth, when Mr. R. W. Hall, Assoc. M. Inst. C.E., read a Paper on " Laying a Water Main under a Tidal Creek " ; Thursday, 22 February, at Southampton, when Mr. M. G. J. McHaffie, M. Inst. C.E., read a Paper entitled " A Brief Survey of Dredging Operations " ; Thursday, 7 March, at Brighton, when Mr. J. L. Savage's lecture on " The Boulder Dam " was read by Mr. D. Halton Thomson, M.A., M. Inst. C.E.

Yorkshire Association.

The following meetings have been held :—Saturday, 17 February, at Sheffield, when Professor J. Husband, M.Eng., M. Inst. C.E., read a Paper on " Modern Motor Roads " ; Saturday, 2 March, at Leeds, when Mr. J. C. Cotton presented a programme of films entitled " The Manufacture of Cast Iron and Concrete Pipes by the Centrifugal Process " ; Thursday, 14 March, at Leeds, when Mr. F. S. Snow, M. Inst. C.E., read a Paper on " Shoring, Strutting and Timbering."